RESEARCH AND DEVELOPMENT IN MANITOBA MINES
TASK 2 - GROUND STABILITY EVALUATION WITH PARTICULAR REFERENCE TO AN EN ECHELON LENSED OREBODY - PHASE II
MANITOBA MINERAL DEVELOPMENT AGREEMENT

CABLE SUPPORT TRIAL AT THE RUTTAN OPERATION - SILL PILLAR -

by

HUDSON BAY MINING & SMELTING CO., LIMITED
in association with
PAKALNIS & ASSOCIATES

DSS File Number: 14SQ.23440-4-9147-5
DSS Contract Serial Number: OSQ84-00330
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CANMET Scientific Authority: Dr. Somchet Vongpaisal

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Work on the Project was Conducted Under the Auspices of the Canada Centre for Mineral and Energy Technology, Energy, Mines and Resources Canada (CANMET) Under the Canada/Manitoba Mineral Development Agreement.

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EXECUTIVE SUMMARY

This study summarizes a cable bolt program that was implemented at the Ruttan operation under the joint sponsorship of Hudson Bay Mining & Smelting Inc., CANMET and the Department of Mines of Manitoba under the Canada/Manitoba Development Agreement. The overall objective was to study the effectiveness of employing cables to support the back of an individual stope.

The 550-20JN sill pillar was supported employing two 16mm diameter cables per hole thereby providing an ultimate support capacity of 52 tonnes per hole. A 2.4m x 2.4m pattern was employed with the individual cables being 10m in length. Approximately 2500m of cable and 1125m of drilling (51mm diameter holes) were required. The sill pillar dimensions were 12m in vertical thickness (width) and the footwall to hanging wall dimension was 14m (height). The intervening sill was located between the 430m and 450m levels below surface. The strike length of the opening below was 29m to a depth of 100m below the pillar. Approximately 110m of stope was open above the sill. Previous sill pillars at the Ruttan operation having similar dimensions had failed.

Blasting of the 550-20JN stope (2nd lift) commenced on April 21 and was completed on July 9/89. Extensometer and tension gauges were employed to monitor the effect of mining on the supported sill. In addition, a seismograph was located on the 430mL in order to monitor the affect of blasting in the vicinity of the cables.

Loads on cables within the sill were estimated to be under seven tonnes with virtually no movement recorded by the wire extensometers. The back was observed to be stable with minimal sloughing. However, the unsupported hangingwall caved to approximately 27m from its original 14m width (FW-HW).

Seismograph records showed that the peak particle velocities were largely affected by the presence of the 450mL drill drive. The peak particle velocities were generally a third of that which would have resulted if the drift was not present. Maximum velocities of 23cm/sec were recorded. A value of 50cm/sec was estimated to cause damage to the cable/grout bond by Stillborg (1984).

It was concluded that the cable bolt trial was a success and it has been adopted by the Ruttan operation as a method of support for sills and stope walls where potential sloughing will be a major problem.

RÉSUMÉ À L'INTENTION DE LA DIRECTION

Le présent rapport résume un programme de boulons de cables exécuté a l'opération Ruttan parrainé conjointement par la Hudson Bay Mining and Smelting Inc., CANMET et le ministère des Mines du Manitoba, dans le cadre de l'entente Canada-Manitoba sur la mise en valeur des minéraux. Ce programme visait globalement à étudier l'efficacité de l'emploi des cables pour soutenir le toit d'un chantier.

Le pilier de toit 550-20JN soutenu par deux cables de 16mm de diamètre par trou ainsi a fourni une capacité de soutien ultime de 52 tonnes par trou. On a employé un système de 2.4m x 2.4m avec des cables de 10m de longueur et a requis approximativement 2500m de cable et 1125m de forage (les trous 51mm de diamètre). Les dimensions du pilier de toit étaient 12m d'épaisseur verticale (largeur) et la dimension du mur au toit était 14m (hauteur). Le pilier de toit intermédiaire s'est trouvé entre les niveaux 430m et 450m au-dessous de la surface. La longueur de l'ouverture en dessous était 29m à une profondeur de 100m au-dessous du pilier. Il y avait approximativement 110m du chantier ouvert au-dessous du pilier de toit. A l'opération Ruttan, il y avait une rupture des piliers de toit précédents qui avaient les dimensions semblables.

On a commencé le sautage du chantier 550-20JN (2^{e} étage) le 21 avril et l'a complété le 9 juillet 1989. On a employé les indicateurs de tension et d'extensomètre pour surveiller l'effet de l'exploitation du pilier de toit soutenu. Aussi, un sismographe trouvé au niveau 430m surveillait l'effet du sautage à proximité des cables.

On a estimé que les charges sur les cables dedans le pilier de toit étaient moins de sept tonnes sans presqu'un mouvement. Des observations ont montré que le toit était stable avec une dilution minimale. Cependant, le toit non soutenu a foudroyé à approximativement 27m de sa largeur originale de 14m (mur à toit).

Des rapports du sismographe ont montré que la présence de l'ouverture au niveau 450m avait un grand effet sur les vitesses de particles maximums. Les vitesses de particles maximums étaient généralement un troisième de ce qui aurait résulté si l'ouverture n'avait pas existé. On a enregistré des vitesses maximums de 23 cm/sec. Une valeur estimée de 50 cm/sec a causé du domage à l'adhérence du cable/cément de Stillborg (1984).

On a conclu que l'essai des boulons de cables a réussi et l'opération Ruttan l'a adopté comme la méthode de soutien pour les piliers de toit et les parois des chantiers ou la dilution potentielle sera un problème majeur.

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The project team acknowledges Mr. Brad Lantz who initiated the study and Dr. Rimas Pakalnis, Edgar Wright, and Dr. Scott Dunbar who conducted the research.

1.0 GENERAL

A cable bolt trial has been implemented at the Ruttan operation of Hudson Bay Mining and Smelting Company Limited. The study is an addendum to a larger program that had been undertaken by Ruttan to develop design guidelines for the mining of open stopes under controlled and predicted conditions (TASK 2). The Work Tasks were commissioned by CANMET under the Canada/Manitoba Mineral Development Agreement. The Task 2 study was completed and reports issued in August of 1989 (CANMET, 1989).

1.1 SCOPE OF STUDY

Hudson Bay Mining and Smelting Company Limited has been commissioned by the Canada Centre for Mineral and Energy Technology under the CANMET/Manitoba Mineral Development Agreement to undertake the following Task:

1. Review the state-of-the-art of an artificial ground reinforcement in mining using cable bolting in Canada and around the world. This project was completed in March of 1989 and the report is included in Appendix I of this study.

2. Install grouted cable bolts in the stope hangingwall or crown pillar and determine their effectiveness in attempting to control the dilution in a large blasthole stope. The sill pillar between the 430m and the 450m (below surface) levels were cable bolted and instrumented. The sill overlies the 20JN stope that has been previously excavated between the 550m and 490m Levels and as part of this study was excavated to the 450m Level. The results are summarized in this study.

3. Verify the influence of the dynamic affects (blasting) on ground stability and cable bolting using the finite element technique. A blast vibration recorder was installed on the 430m Level in order to monitor the excavation of the 20JN stope. Appendix II describes the finite element modelling of velocity records of blasts detonated within the 20JN stope.

1.2 INTRODUCTION

The general background with respect to the Ruttan operation is summarized within the TASK II report and the reader is referred to this report for greater detail (CANMET, 1989).

The Ruttan Mine, a 6,000 tpd underground operation, is located in Northern Manitoba, 760 km North of Winnipeg and 20 km East of the town of Leaf Rapids. The Ruttan copper-zinc orebody is a multi-lensed, steeply dipping (70°) en echelon deposit. Individual lenses have a maximum strike length of 350 meters with widths varying from 7 to 61 meters.

The mining method is open stoping with delayed backfilling which extends from surface to 860 meters below surface. Production commenced by open pit in April, 1973 and continued until December, 1980, whereafter production has been entirely from underground. The mine has been owned and operated by the Hudson Bay Mining and Smelting Company since July, 1987.

Historically, stopes at Ruttan whose sill pillar dimensions were less than 1:1 had failed. This refers to the pillar width (W), recorded as the vertical pillar thickness, to the height, recorded as the distance between the footwall and hangingwall. In April of 1989 the second lift of the 550-20JN stope was to be mined, thereby exposing a sill pillar of 12m vertical thickness with a stope dimension of 14m (F/W to H/W). Based upon previous case histories and numerical assessments it was shown that the pillar would most likely fail. It was decided to cable bolt the 550-20JN stope sill pillar above the 450mL. The concern with the sill pillar failure were due to:

* dilution of the ore by backfill and waste rock located in stopes previously mined out above the sill (mined to the 320m Level);
* drawpoint plugs and excess secondary blasting;
* loss of access to adjacent stopes on the 450m Level;
* potential caving of the stope wall contacts.

This report summarizes the cable bolt trial implemented on the 550-20JN sill with respect to the overall effectiveness of this form of support. This was determined visually as well as through instrumentation.

2.0 CABLE BOLT PROGRAM

Approximately 2300m of 16mm diameter wire cables (26 tonne) were installed in the 550-20JN sill. The cables were installed on a 2.4m x 2.4m pattern employing two cables per hole. A total of 1124m of drilling were required and comprised 114 holes each approximately 10m in length. The individual holes were 10m in length. Two cables per hole were employed to develop a strength of 52 tonnes.

2.1 550-20JN STOPE

Figure 1 shows the longitudinal, plan and typical vertical section for the 550-20JN stope. The F/W and H/W is comprised of intermediate to acid sediments with the sill pillar consisting of massive sulphides. Table 1 (CANMET, 1989) summarizes the intact rock strength parameters as determined in TASK 2. Generally the rock mass was observed from drill hole data and mapping on the 450mL drill drive. The sill pillar has an average rock mass rating (CSIR) of 85%. The RQD was estimated at 80%.

Table 1

INTACT ROCK STRENGTH PARAMETERS
(TASK II-CANMET,1989)

Rock Type	Uniaxial Compressive Strength (MPa)	Poisson's Ratio	Modulus of Deformation (GPa)	Number of Samples
Semi-Massive Sulphide-Chlorite	70 ± 24	0.23 ± 0.05	66 ± 19	13
Massive Sulphide	89 ± 37	0.2 ± 0.08	88 ± 37	39
Basic Dyke	202 ± 66	0.2 ± 0.07	81 ± 14	10
Acid Sedo.	135 ± 43	0.19 ± 0.07	77 ± 19	6
Quartzites	113 ± 49	0.12 ± 0.06	54 ± 23	4
Chlorite Schists	49 ± 11	0.34 ± 0.12	50 ± 17	14
Chlorite Talc Schists	25 ± 14	0.33 ± 0.09	39 ± 9	13

Two joints sets were observed, one horizontal and spaced approximately at 1m intervals and a vertical dipping set that is perpendicular to the trend of the orebody with a 0.3m spacing. Both sets are considered to be continuous with respect to the sill pillar dimensions. A lesser occurring steeply dipping random set was also observed. The area was dry with the individual joints surfaces being recorded as being planar smooth.

The 550-20JN Stope is located approximately 25m in the H/W of 490-19K Stope which was excavated prior to the mining of the second lift of 550-20JN. The 490-19K stope extends from the 490mL to 20m above the 430mL, Figure 1. The 20J stope is mined and filled to the 320mL whereas the 19J stope is mined and filled to the 390mL. The intervening sill between 390m and 370m levels had failed in the vicinity of the 20J Stope, Figure 1.

The ultimate geometry that was excavated as part of this study is shown in Figure 1 and is as follows:

A Stope Dimensions

* strike length = 29m
* stope width = 15m
* ultimate stope height = 100m
  a) first lift 550mL to 490mL
  b) second lift 490mL to 450mL

B Sill Pillar Dimensions

* strike length = 29m
* pillar width (F/W to H/W) = 14m
* pillar vertical thickness = 12m (450m to 430mL)

2.2 PATTERN

The cable bolt pattern selected was 2.4m x 2.4m employing two 16mm cables per hole (52 tonne) and is shown in Figure 2. It was determined after reviewing the state of knowledge as it presently exists concerning the application of non-tensioned cable bolts. This review is presented in Appendix I of this report. The following approaches were employed:

A Bharti (1983), based upon observations at the Strathcona Mine of Falconbridge Ltd., concluded that the height of failure is limited to 0.2 to 0.25 x stope width. This would dictate that a 14m wide stope would require to support a block approximately 3.5m in vertical height. This would yield a pattern of 2.2m x 2.2m cable bolts (52 tonne).

B Potvin (1988) has related cable bolt density to the block size/hydraulic radius as shown in Figure 3 and reviewed in Appendix I. The 550-20JN sill pillar can be characterized as shown in Figure 3. This would indicate that a 2.6m x 2.6m pattern is recommended (employing two cables/hole)

C Table 2 is an excerpt from Appendix I and summarizes a survey of ca bolt practices performed by the author. It generally shows that where two cables per hole are employed (52 tonne) a pattern in excess of 1.8m x 1.8m is practised.

2.3 CABLE BOLT INSTALLATION PROCEDURE

Installation procedures are as summarized in Appendix I. Generally 51mm (2 inch) diameter vertical boreholes, 10m long, were drilled upwards on 2.4m centres into the sill pillar from the 450m Level. Some inclined holes were also drilled from an adjacent footwall drift. Pre-cut 1.6cm (5/8") x 2 cables were installed in each hole. Spring clips were attached to the end of the cable to hold it within the hole. The cable was cleaned prior to installation. A 0.95cm (3/8") diameter breather tube was attached 15cm from the end of the cable using electrical tape. Several wraps of tape were used at various locations along the length of the cable to secure the tube. The cable was inserted into the hole until approximately 2m of cable was left exposed. A 1.9cm (3/4") grout tube was attached with tape approximately 2m upwards from the collar. The cable was then pushed manually into the hole. The collar of the hole was sealed with burlap and wooden wedges. The grout tube and breather tuber were cut about 3m from the collar. This is shown in Figure 4.

The grout tube was attached to the feeder hose of the grout pump. A water cement ratio of 0.4 by weight was employed. A Type 30 Portland Cement (high early cement) was employed in order to develop maximum strength in the shortest duration (i.e. three days to achieve over 50% maximum strength). The cement was pumped into the hole until it discharged from the breather tube. A Spedel Series 6000 pump and B3100 mixer were employed. A two man crew was employed for installation. Approximately one hole was installed within a 45 minute interval (not including drilling).

2.4 INSTRUMENTATION

Three types of instruments were installed within the sill pillar:

* wire extensometers (IMS);
* vibrating wire stressmeters (Geokon);
* tension gauges (TENSMEG-70).

Table 2: SURVEY OF CABLE BOLT PRACTICES

	MINE	APPLICATION	BOLT LENGTH	PATTERN	HOLE SIZE	GROUT	BOLT SIZE/TYPE	YIELD LOAD	OTHER
1	Brunswick Mining and Smelting Brunswick #12	Cut & Fill 4.9m lifts	17.7m	1.8m x 1.8m (uphole:)	57mm(2-1/4")	high early cement(HE) expansive additive water/cement = 0.4	2 x 16mm (5/8") strand	53 tonne	6mm (1/4") I.D. breather tube
2	Falconbridge Strathcona	Cut & Fill 3m lifts, 3m overlap areas with RQD<50%	17m	1.8m x 1.8m	54mm(2.1")	portland cement-norm water/cement = 0.4 cable buttons used	1 x 16mm strand	27 tonne	6mm breather tube 12.5mm (1/2") grout tube
3	INCO Thompson	Cut & Fill	6.1m, 12.2m	1.2m x 1.2m(1 cable) 2.4m x 2.4m(2 cables)	54mm	portland cement-norm water/cement = 0.45	1 or 2 x 16mm strand	27 or 54 tonne	9.5mm(3/8") breather with Mono pump 12.5mm breather with Spudel pump
4	Con Mine Yellowknife	Cut & Fill 3m lifts + 4.6m H/W	10m, 23m	2.2m x 2.4m	57mm	portland cement-norm water/cement = 0.33	2 x 16mm strand	58 tonne	6mm breather tube
5	New Broken Hill Australia	Cut & Fill 2.4m lifts	20m	2m x 2m	57 mm	high early cement bleeding additive water/cement = 0.51	2 x 16mm strand	53 tonne	3mm breather tube
6	Homestake S. Dakota	Open Stope H/W support	15m	3m x 3m	57 mm	portland cement-norm water/cement = 0.5	1 x 16mm strand	29 tonne	buttons used
		Cut & Fill	18.3m	1.8m x 1.8m	64 mm	portland cement-norm water/cement = 0.5	2 x 16mm strand	58 tonne	12.5mm breather tube buttons used
7	LynnGold MacLellan	Open Stope H/W & F/W	6m	2.4m spacing (2 holes) every 20m sub-level	54 mm	portland cement-norm water/cement = 0.45	2 x 16mm strand	58 tonne	holes drilled from sub-levels at wall contact
8	Placer Dome Campbell	Open Stope H/W support	15m	2.4m spacing (3 holes)	51 mm	portland cement-norm/HE water/cement = 0.4	1 x 16mm strand	29 tonne	holes drilled from sub-levels at wall contact
		Cut & Fill 3m lifts	15m	2.4m x 2.4m	51 mm	portland cement-norm/HE water/cement = 0.4	1 x 16mm strand	29 tonne	support dead weight equivalent in 0.2 m exposed width(F/W - H/W
9	West Coast Mines Tasmania	Open Stope H/W support	20m	1.8m x 1.8m (perpendicular to H/W)		portland cement-norm water/cement = 0.5	7 x 7mm wire	45 tonne	holes drilled from sub-levels breather tube 12.5mm, 19m grout tube buttons used
10	Outokumpu Oy Finland	Open Stope H/W support	50m to 15m	1 bolt per 25 sq. meters to to 1 bolt per 50 sq. meters	41mm	portland cement-norm water/cement = 0.5, 0.45	2 x 15mm strand	50 tonne	Fan shaped from F/W and H/W access with spacing 3m to 10m
					64mm	portland cement-norm water/cement = 0.5,0.3	1 x 15 mm strand	25 tonne	Fan shaped from F/W and H/W access with spacing 3m to 10m
11	Mamberget Mines Sweden	Open Stope F/W support	25m to 15m	2.5m along strike (1 hole) every sub-level (20m vertical)		portland cement-norm water/cement = 0.5, 0.45	1 x 36 mm strand	90 tonne	drilled at F/W contact from sub-level RQD of F/W 20 to 60%
12	Mount Isa Mines Ltd. Australia	Open Stope H/W support	9-12m	2m spacing (3 holes) every sub-level (30m vertical)	57-70 mm	portland cement-norm water/cement = 0.5	2 x 15mm strand	50 tonne	holes drilled from sub-levels at H/W contact, first 2m debonded
13	Westmin H-W Mine	Cut & Fill	15m Open Stope	2m x 1.5m	51mm	portland cement-norm	2 x 15mm strand	50 tonne	
14	Eldorado Beaver Lodge	Cut & Fill	15m	1.8m x 2.4m	57mm	portland cement-norm bleeding additive	1 x 13mm strand		Spudel grout pump employed

Tension gauges were installed on cable bolts at two locations. The first tension gauge was installed within the slot area in an uphole on the 450mL drill drift, Figure 5. A stressmeter was also installed in the vicinity. The wire cables were protected within steel casing, however, were destroyed with the onset of blasting.

A second tension gauge was installed within the sill pillar from 20J #1 Drawpoint on 430mL, Figure 5. This consisted of an 18.3m cable bolt inclined at -35 degrees to intercept the slot area of the stope below. Tension gauges were installed at 7.9m, 12.8m and 16.6m from the collar (430mL). These will subsequently be referred to as TENSMEG gauge #1,#2 and #3 respectively. Adjacent to this cable bolt, in the plane of "ring Z", three wire extensometers were installed in a hole inclined at -32 degrees, Figure 5. The extensometer anchors were at 7.9m, 12.8m and 16.6m from the collar. These will be subsequently referred to as extensometer gauges #1,#2 and #3 respectively. This extensometer hole was employed to verify the TENSMEG results. Both instruments were located so as to measure movement of the sill approximately two,four and six meters above the back of the 450mL. Extensometer movements are relative to the initial reading.

A 13.5m wire extensometer hole was drilled on ring 6 at +23 degrees from the 21JN Access Drift in the footwall on 450mL. The single anchor was placed at the bottom of the hole. This hole served as a sloughmeter paralleling the base of the sill pillar, Figure 5.

Stressmeters were placed into holes drilled within the back of the sill on 450mL in the vicinity of the slot, however, due to blasting within close proximity the installation was lost.

2.4.1 Instrumentation Procedure

The vibrating wire (Geokon) stressmeters and wire extensometers (IMS) have been described in detail in Canmet (1989). The TENSMEG-70 gauge is unique to this project and a discussion on installation is summarized in Appendix III. The tension measuring gauge "TENSMEG-70" was developed at Laval University (Quebec city, Canada). It is based on a thin 2.4m resistance wire (0.25mm) folded back three times between two end anchors for a total gauge length of 0.7m. The anchors are pretensioned and glued onto the cable bolt.

Subsequent tensioning of the cable bolt is recorded by a change in electrical resistance within the wires. A Vishay strain readout unit was employed. Values were recorded after each blast.

3.0 RESULTS

Table 3 is a summary of sill pillar dimensions at Ruttan.

Table 3

SILL PILLAR DIMENSIONS - RUTTAN

Stope	Dimension Vertical (Width)	FW-HW (Height)	Depth	Rock Type	Comments (Width/Height)
430-21K	12.2m	6.5m	370m	Massive Sulphide with Schist	Stable (1.9)
430-20K	17m	11.5m	370m	Massive Sulphide	Stable (1.5)
430-20J	13m	16m	370m	Massive Sulphide	Failed (0.9)
550-2D	20m	25.5m	430m	Massive Sulphide with Schist	Failed (0.8)
730-4D	21m	24.5m	660m	Massive Sulphide with Schist	Failed (0.9)
550-20J	12m	14.5m	430m	Massive Sulphide	Test Stope (0.8)

Based upon past observations and that the 550-20J sill is open approximately 100m below the sill and approximately 110m above the sill to the 320mL for the full strike length of the 20J stope it was recommended that this sill should be supported.

The blast history is shown in Figure 6. Blasting commenced on April 21/1989 and was completed on July 9/1989. The history is shown in Table 4.

Table 4

STOPE BLAST HISTORY - 550-20JN

Blast Number	Date	Details
1x	April 21/89	Cone blasted to 1048mL
2x	June 8	Slot blasted to 1050mL
3x	June 15	Slot blasted to 1058mL
4x	June 30	Rings 6-13 + slot blasted
5x	July 9	Rings 1-5 blasted

Figures 7 and 8 show the TENSMEG and wire extensometer readings. Generally the two TENSMEG gauges located above the 450 drill drive showed that the cables were under increasing load prior to Blast 5x (60 to 80kN).

Instrumentation was in place after the smaller slot blasts had been initiated. The wire extensometers measured under 0.1mm of elongation during the above ring blasts. This is well within the error of the instrument ($\pm$ 0.025cm). Gauge #1 on both the TENSMEG and wire extensometer exhibited no load or displacement. This gauge was located within the footwall of the exposed sill, Figure 5.

It was further observed that after Blast 5x that the TENSMEG readings were in less tension than prior to the blast. The trend was equally observed in the wire extensometers. This was possibly due to the relaxation of the pillar (yielding) and consequently less load transfer onto the cables. Gauge #3 showed a constant load of approximately 70kN (7 tonnes) whereas Gauge #2 showed a decreasing load to below 0kN approximately eight months after the blast.

Visual observation had shown that the sill pillar had not deteriorated. This was evident through the existence of plates located at the collar of the cables. A vertical section of 550-20JN is shown in Figure 9 whereby failure of the unsupported hanging wall was observed to the dimensions shown. It was observed that the cable bolted sill was stable and sloughing (1.5m) was observed above the 450mL in the proximity of the sill which was undercut due to the hangingwall slough, Figure 9.

An addendum to this study is found in Appendix II whereby blast vibration records for Blast 3x and 5x are employed to verify a finite element dynamic model. This model has been developed by D. Gordon Dunbar and Associates (1988). A seismic recorder (Instantel DS477) was placed on the 430mL, Figure 6.

Conclusions of the above blast vibration study had shown that simulated vibration records to that actually recorded was possible.

It was further shown that due to the presence of the 450mL drill drive separating the sill and the stope blasts, one results in much lower peak particle velocities in the vicinity of the cables. It was estimated that without the intervening drift (450mL) that the peak particle velocities would be approximately three times that which was observed.

The maximum peak particle velocities recorded were 4.8cm/sec for Blasts 3X and 22.6cm/sec for Blast 5x. The individual blast records and analysis are discussed in detail in Appendix II. Stillborg (1984) has shown through pull-out tests that if reinforced rock is subjected to dynamic loading from a nearby production blast, the load bearing capacity of the cable will only be affected when the particle velocity exceeds 50cm/sec at the cable. Based upon the vibration levels recorded during the study and the instrumentation records it was determined that blast damage to the cable/grout bond would be minimal.

4.0 DISCUSSION AND CONCLUSION

It was concluded that the cable bolt program implemented for purposes of sill pillar stability for the 550-20JN Stope was highly successful. The 2.4m x 2.4m cable bolt pattern (two 16mm cables/hole) maintained the stability of the intervening pillar. The vertical dimension of the pillar with respect to the opening width below is 12m/14.5m or 0.8. Based upon similar dimensions, stress levels and rock qualities observed at Ruttan for sill pillars, it was concluded that this pillar will ultimately collapse if not supported. The hangingwall in the vicinity of the sill had sloughed approximately 12m thereby increasing the FW/HW dimensions of the exposed sill. No sloughing of the supported sill was observed.

Instrumentation had shown that the cables were loaded in minor tension during the stope excavation process, however, it was not reflected in actual significant elongation of the adjacent wire extensometers. The loading was under seven tonnes on the instrumented cable. Thereafter it appeared that relaxation of the cable had occurred through shedding of the load to adjacent areas.

Blast vibration records had shown that the vibration levels were well under the critical values required to cause damage to the cable/grout as defined by Stillborg. The existence of openings was shown to greatly reduce vibration magnitudes as the blast pulse must travel around individual drifts.

The cable bolt program was considered to be a success and has been implemented for all work areas at Ruttan where it is considered that sill pillar and/or hangingwall slough will be a major problem.

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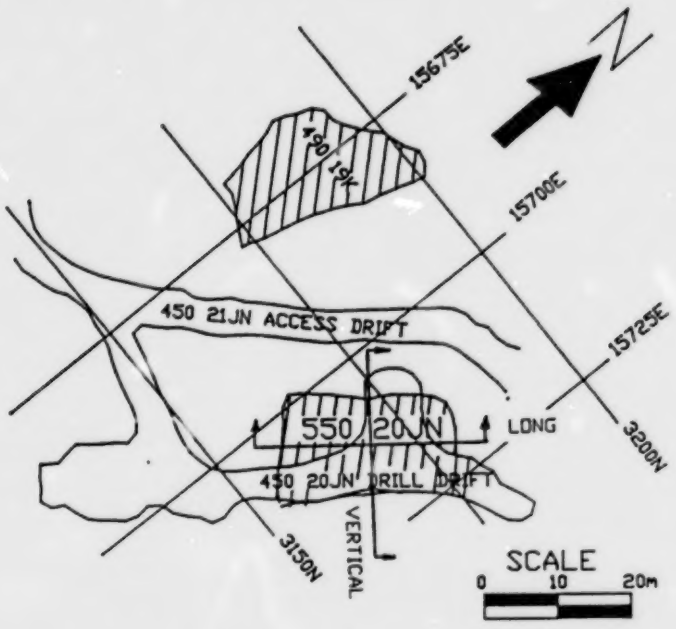
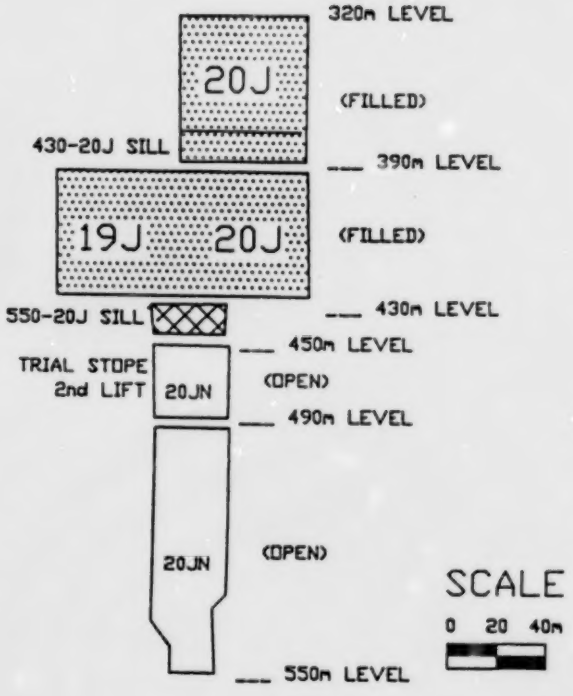
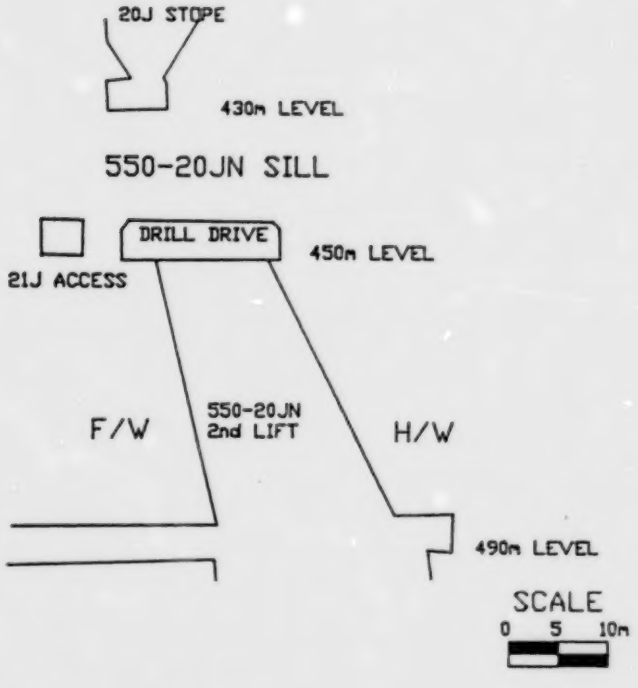


450m LEVEL PLAN

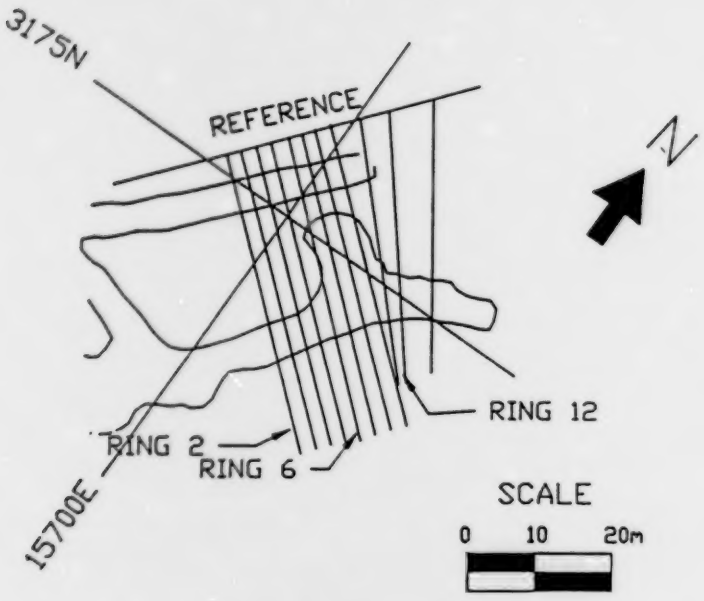


LONG SECTION

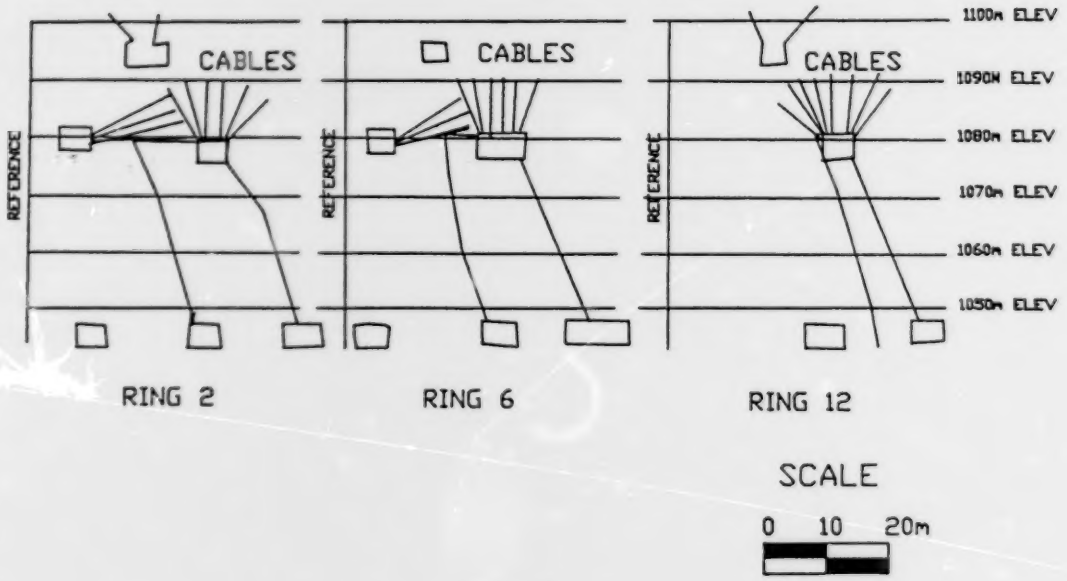


VERTICAL SECTION

Figure 1: STOPE DIMENSIONS 550-20JN



450mL PLAN - CABLE BOLT RING LAYOUT



CABLE BOLT SECTIONS

Figure 2: CABLE BOLT PATTERN

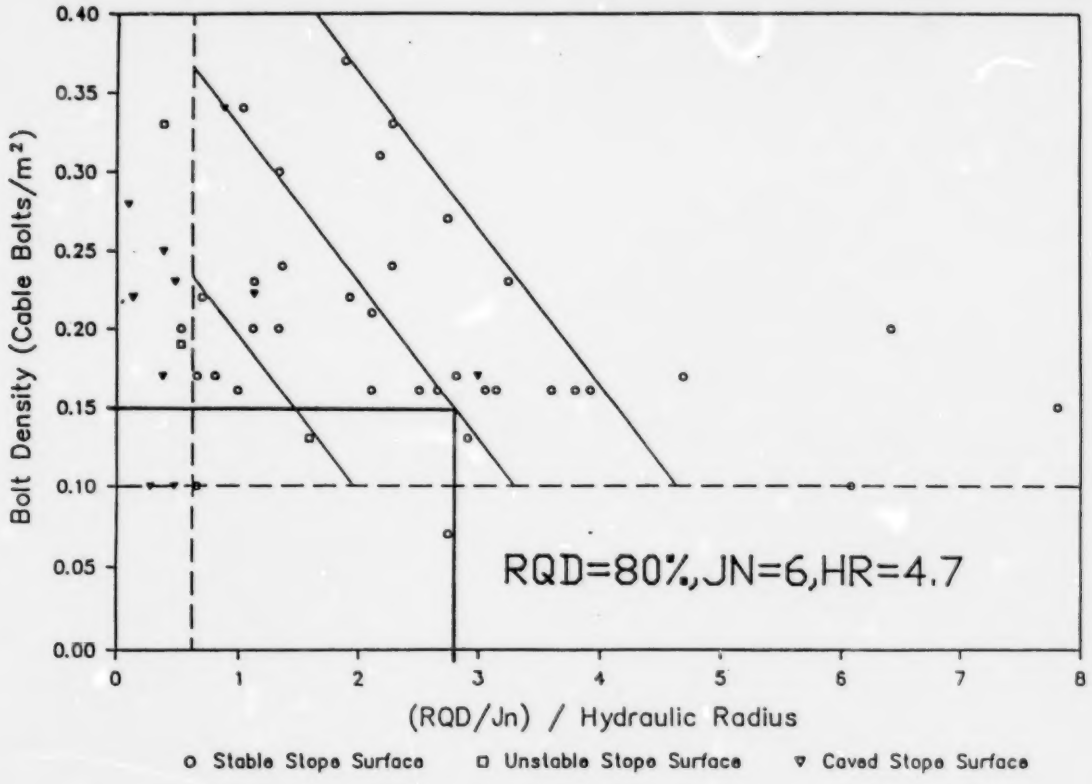


Figure 3: DESIGN CHART CABLE BOLT DENSITY (POTVIN, 1988)

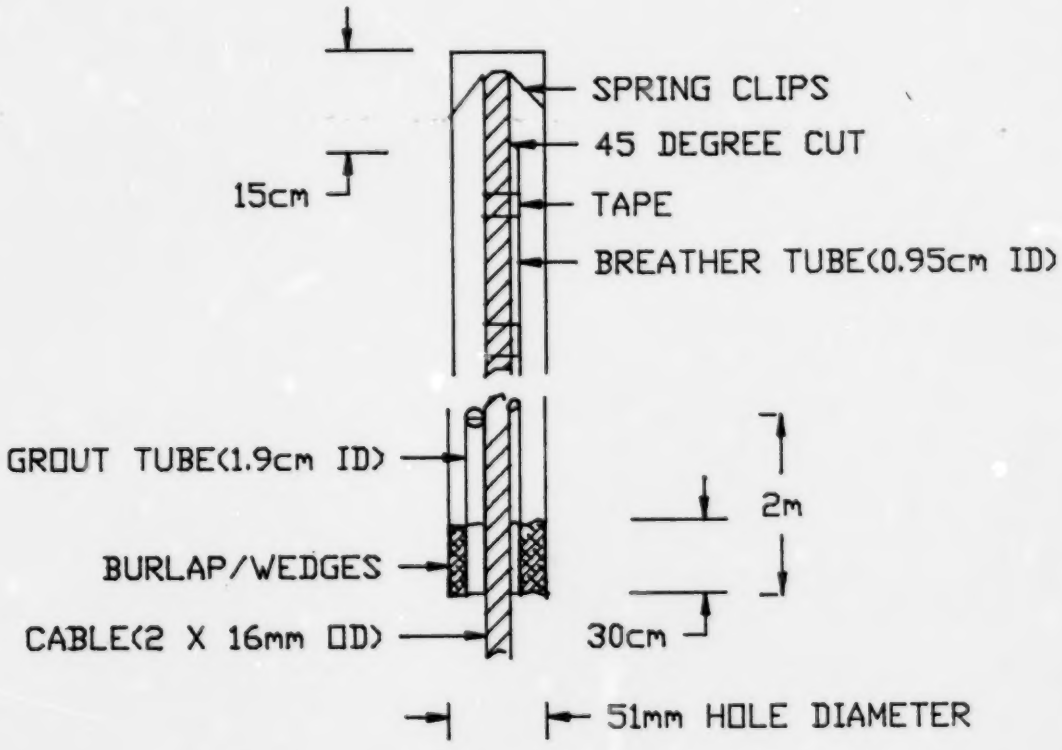
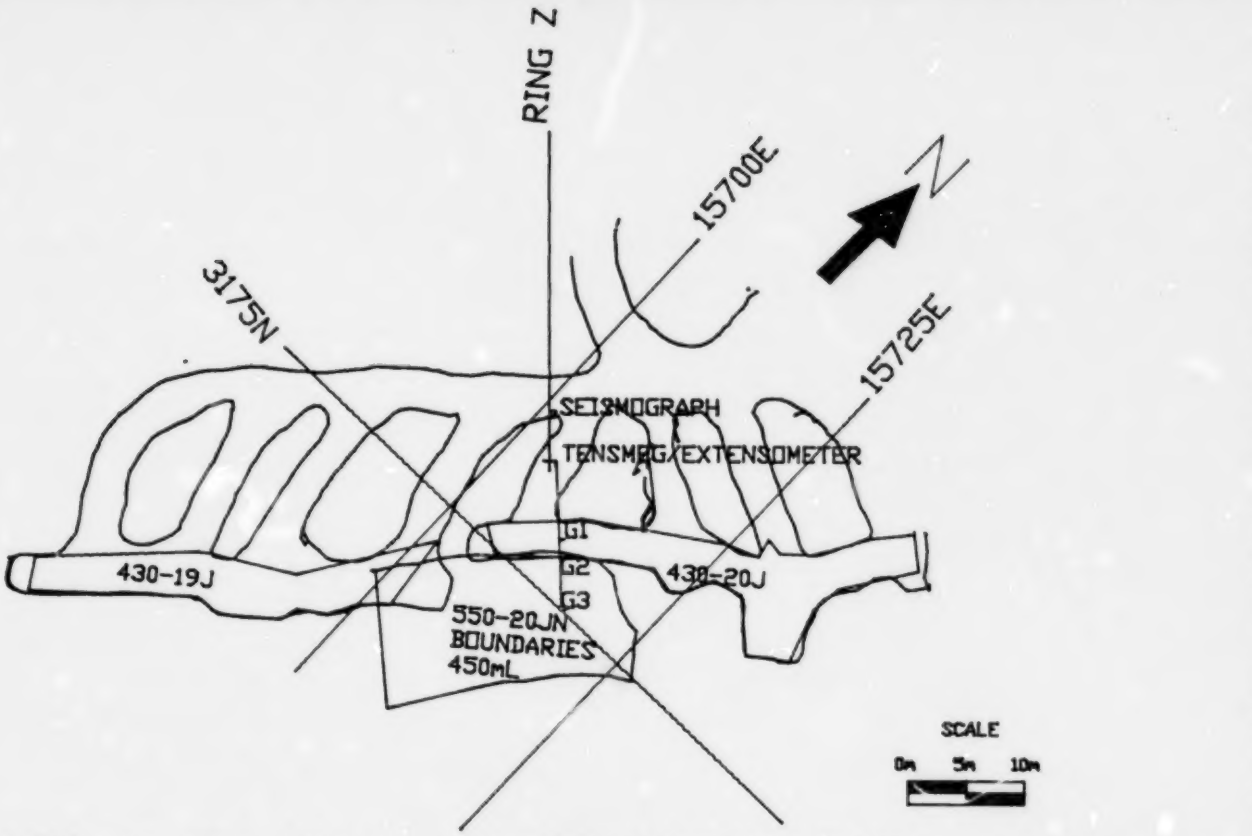
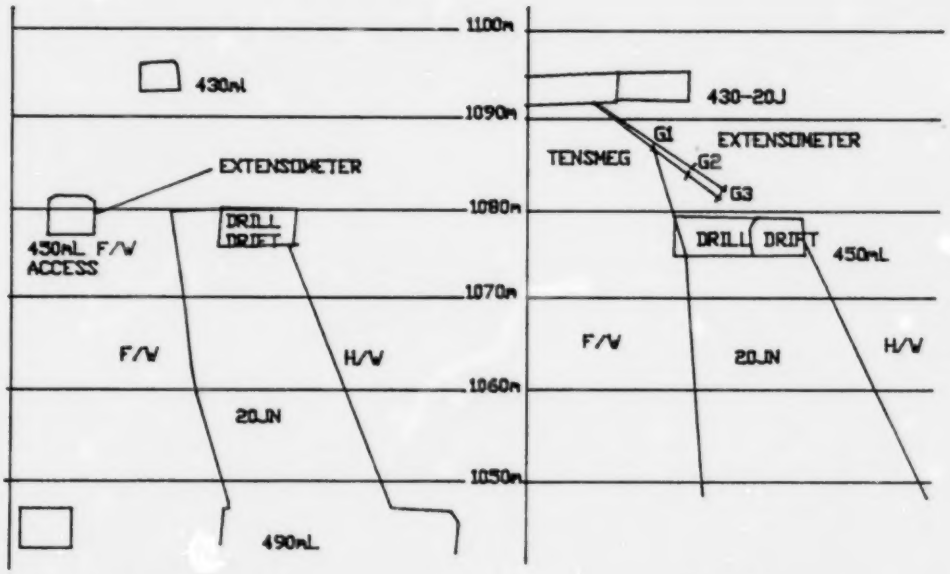


Figure 4: CABLE BOLT INSTALLATION



430mL PLAN - INSTRUMENTATION LOCATION



RING 6

(refer Figure 2)

RING Z

SECTION

Figure 5: INSTRUMENTATION

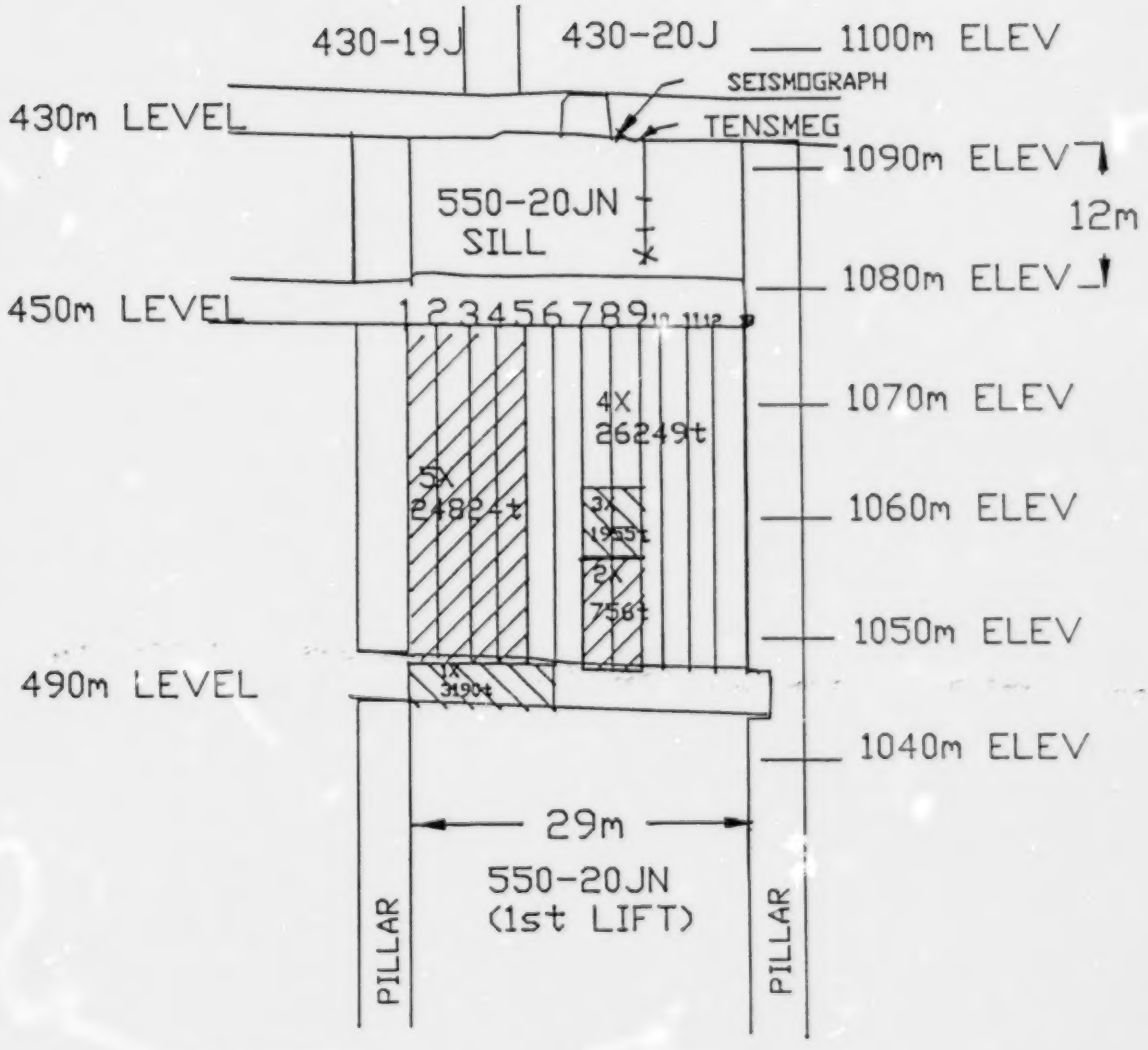


Figure 6: BLAST HISTORY - LONGITUDINAL 550-20JN

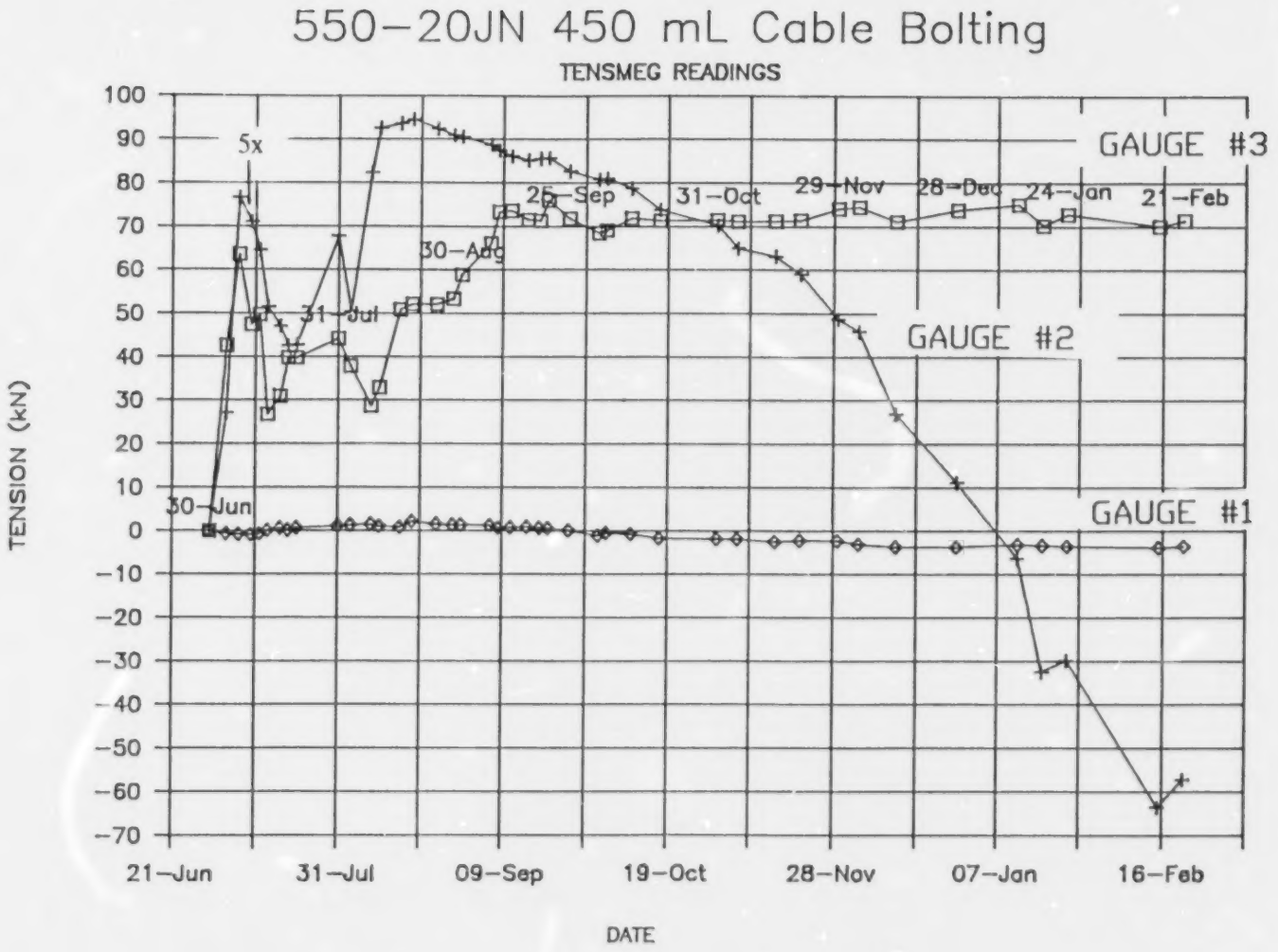


Figure 7: TENSMEG-70 READINGS

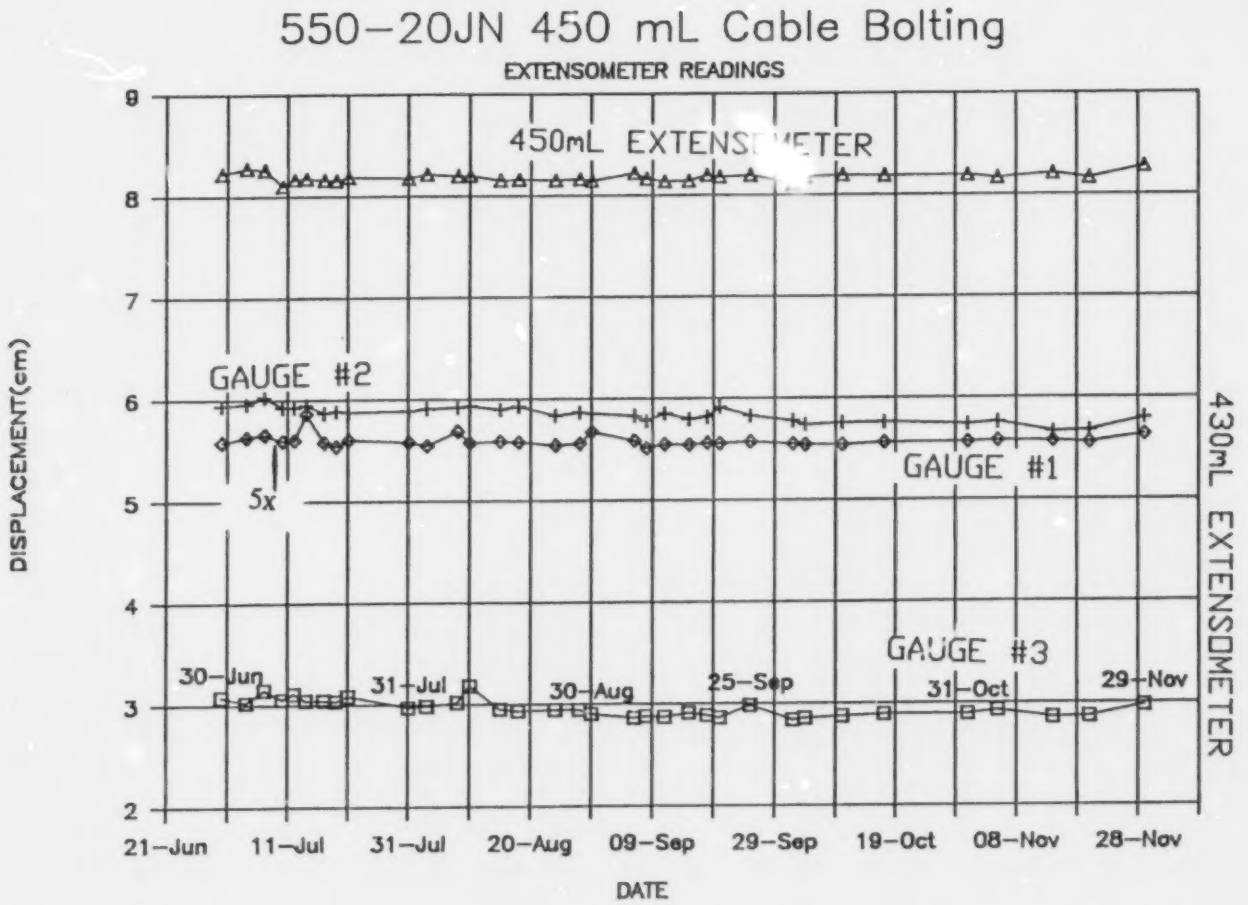
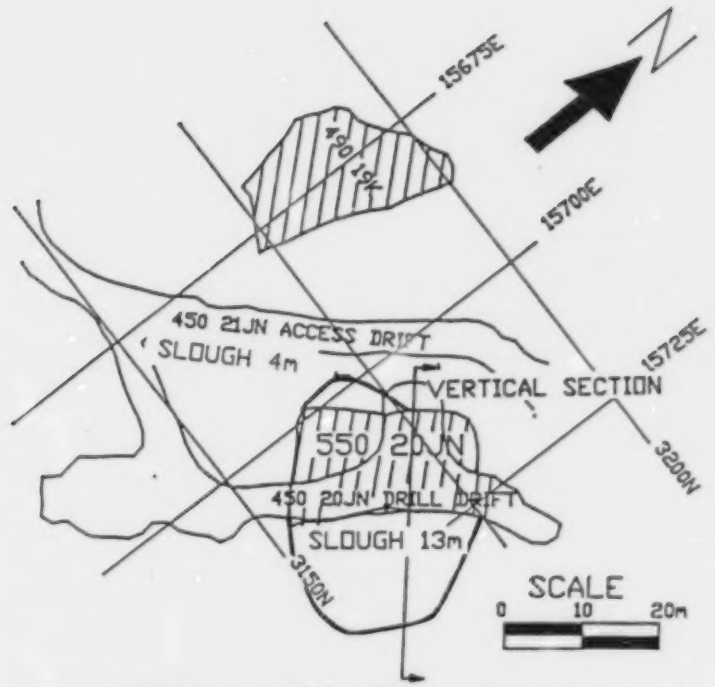


Figure 8: EXTENSOMETER READINGS



450m LEVEL PLAN

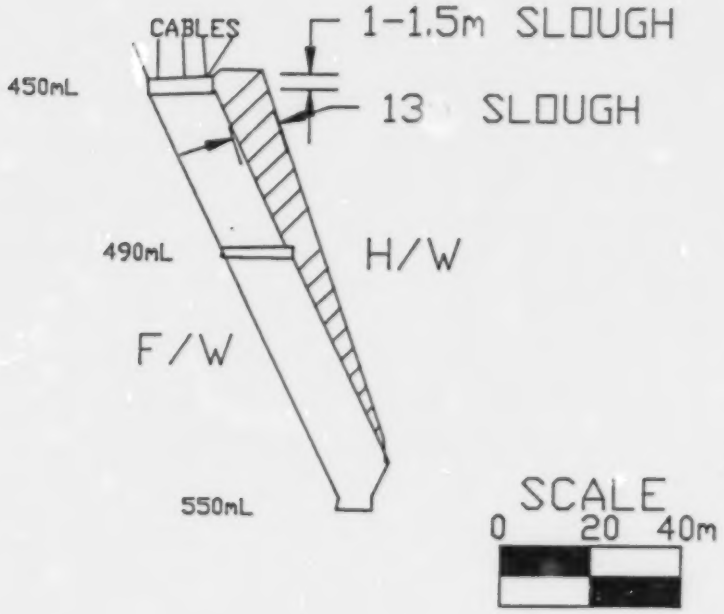


Figure 9: VERTICAL SECTION 550-20JN

COMPLETION OF BLASTING (AUG/89)

COMPLETION OF MINING (DEC/89)

Plate 1: PHOTOGRAPHS OF BACK OF 550-20JN

APPENDIX I

Cable Support Practice in Open Stope Mining

R & D IN MANITOBA MINES
TASK 2 - GROUND STABILITY WITH REFERENCE TO AN ECHELON LENSED OREBODY - PHASE II
MANITOBA MINERAL DEVELOPMENT AGREEMENT

PROJECT: CABLE SUPPORT PRACTICE IN OPEN STOPE MINING

BY
HUDSON BAY MINING & SMELTING CO.
&
PAKALNIS & ASSOCIATES

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March 1989

ABSTRACT

This report evaluates the state of knowledge as it exists presently concerning the application of cable bolts (non-tensioned) in open stope operations. It evaluates present practices and identifies areas requiring further work for purposes of developing guidelines for the implementation of cable support. This study is part of a larger program whereby cables will be installed at the Ruttan mine within a sill pillar and subsequently monitored as the stope is mined.

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1.0 INTRODUCTION

The reinforcement of rock in metal mines using long, flexible cables was introduced in Canada/Australia in the early 1970's. The concepts of rock anchoring are derived from the civil industry whereby using tensioned cables is the norm. Pre-tensioning techniques have been described by Hobst (1983) as a cumbersome and time consuming process. It was realized that pre-tensioning of a fully grouted cable would only influence the rock mass within the immediate back and that on subsequent lifts in a cut and fill operation that the effects of the pre-tensioning would be lost (Fuller, 1983). The concept of employing a passive support (cable dowels) whereby the cable is placed into tension once the rock mass deforms is employed. The relaxation which develops provides the tension. This method of ground support relies on the ability of the rock to partially support itself. This report investigates the use of cable dowels (bolts) for purposes of support in an open stope environment.

The benefits that result from cable support as applied to open stoping result in augmenting the rock quality of the wall or structure to be supported. This results in lower dilutions, increased extraction and added safety.

The design parameters that will be discussed and have the greatest implications on designing a successful cable support program will be classified as follows:

a) Bond Strength to achieve maximum cable/grout/hole strength;
b) Installation
   - procedures
   - inclination of cables
   - pattern;
c) Practices at other mine operations;
d) Cable specifications.

2.0 BOND STRENGTH

The success of a cable bolt program is largely dependent upon the grout bond between the cable and the drill hole. Upon tensioning of the cable due to relaxation of ground in the vicinity, it is required that this movement be transferred to the cable without breakage of the grout bond. The cable tensile strength generally will exceed 26 tonnes (15.2mm) whereas a good quality grout will have an ultimate bond strength of only 4.2MPa (Hobst, 1983). Therefore in order to ensure that the cable will break prior to debonding, one must ensure that an optimum grout mix, installation practice and bond length be achieved.

2.1 BOND LENGTH

In mining applications the entire cable length is generally grouted. The required bond length or the minimum length of cable that must be grouted in order to ensure that pull-out will not occur can be calculated as follows:

a) $P = \pi D \times L \times S$ (eqn. 1)

where P: Tensile Pull-Out Force
D: Diameter (D1) of hole for grout /bore hole bond
Diameter (D2) of cable for grout/cable bond
S: Cohesive strength of grout
L: Bond Length
(as per Figure 1)

The ultimate cohesion of the grout is calculated as being 1/10 of the unconfined compressive strength of the grout. A good quality grout will exhibit a compressive strength of 42MPa. Coates (1970) refers to the maximum working value of 2.45 MPa for bond strength employing a Factor of Safety of 1.75. Hobst (1983) and Fuller (1983) has shown that anchorage in strong rocks is dependent upon the grout/steel bond which is the weakest component in the overall bond length. The cohesion between the grout and steel is a factor of:

a) adhesion - physical bonding between surface of steel and grout;
b) mechanical interlocking - rolled on ribs, threads, buttons etc on the cable;
c) friction - roughness of steel surface and interaction of grout/steel.

Hobst (1983) found that failure of the grout/steel occurs at the top end of the bond length and deteriorates to the root of the bolt. Upon stressing the anchor, adhesion comes into effect at the collar while at the root end, it remains unexploited. When the adhesion is overcome in the collar area, a slip takes place and most of the stress is

gradually transferred to deeper sections of the bolt while at the collar end, only the small effect of friction remain. This was confirmed by L.Horst and Ballivy (1983) in loading tests. Figure 2 shows various modes of failure as observed by Jeremic (1983). It is expected that the mode of failure in situ would resemble that of Figure 2c for stronger rock materials. Figure 3 shows the effect of root configuration on developed stress.

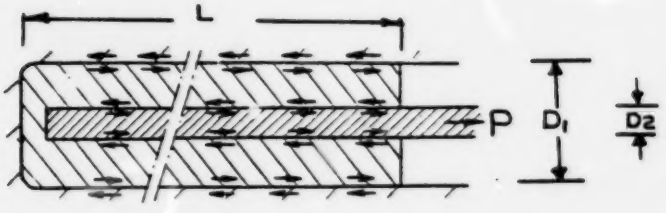


Figure 1: Relationship between bond length and pull-out strength.

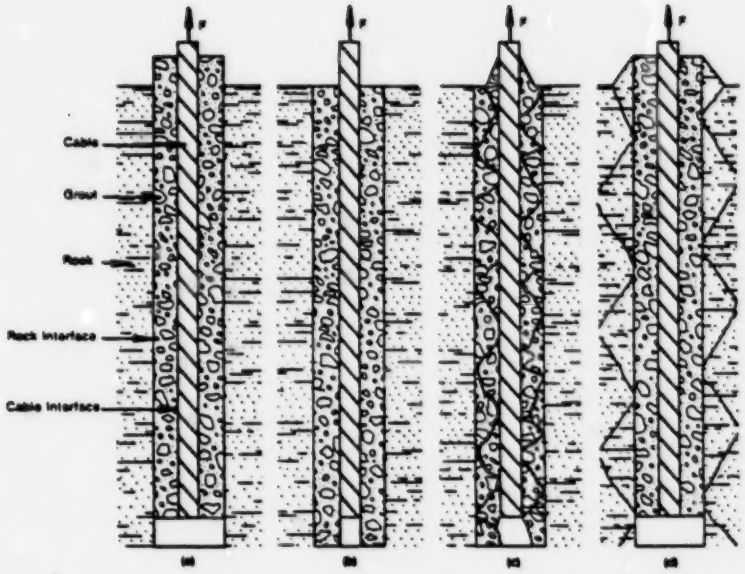
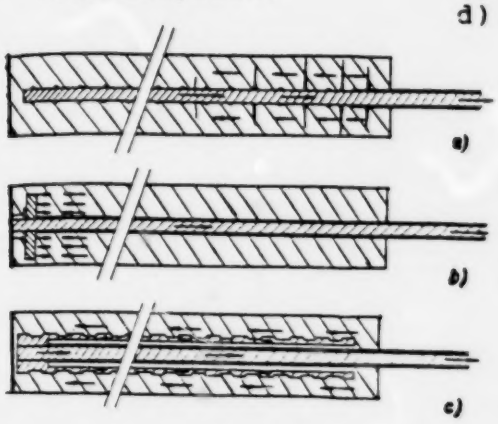


Figure 2: Cable bolt failure modes: a) grout/rock bond
b) grout/cable bond
c) grout
d) rock

(Jeremic,1983)



Basic types anchor fixing involving a concrete root, and the stress occurring in the root
a) — gradual stress development caused by tension, *b*) — stress caused by concentric compression at base, *c*) — shear and compression stresses distributed over a larger area

Figure 3: Root configuration and effect on developed stress.

Based on experimental work, Brown (1970) suggested the following rules for a deformed, Figure 3c, cable:

Sound rock	Anchor length (AL) = 30 x diam. of cable
Fissured rock	AL = 40 x diameter of cable
Altered rock	AL = 60 x diameter of cable

Coates (1970) suggests that a maximum cable/rod adhesion of 2.4MPa be employed. Ballivy (1983) has shown through testing and modelling that 90% of the strength of the cable is achieved when the AL is 16 x cable diameter. He concluded that the strength of grouted cable is related to the bond length until the grouted length exceeds 16 x cable diameter.

The implications of this on fully grouted cable bolts as practised in the mining industry are:

a) On the length of the grouted cable that must be embedded for anchorage. Stillborg (1986) recommends that this length exceed 1m.

b) On the ratio between grout bond length and the weight of the failed block. A block of ground may slip off a cable due to insufficient grout strength as may be the case in the hanging wall of a stope.

It was concluded at a panel discussion at the International Symposium on Rock Bolting in Abisko, Sweden (1983) that pull- out tests will give no information concerning the condition or existence of grout exceeding a bond length of 20cm. This is based upon observations/testing whereby a 20cm length of good quality grout is sufficient to withstand the pull- out force that is applied. It is therefore critical that an optimum grout mixture be employed in order to achieve the maximum pull-out strength of the cable.

2.2 CEMENT GROUT COMPOSITION

The grout generally consists of Portland cement and clean water. The water must be of good quality and must not have a high content of sulphate (more than .1%), chloride (more than 0.5%),sugars or suspended organic matter. Water which is suitable for drinking can be employed in most instances (Littlejohn, 1976). Portland cement is further classified as:

a) Type 10: Normal. This is a general-purpose cement. It is used where the special properties of the other types are not required.

b) Type 20: Moderate. This cement is used where precaution against moderate sulphate attack is important.

c) Type 30: High Early Strength. This provides high strengths at an early period, usually a week or less. Lower water/cement mixes of Type 10 cement will yield similar results.

d) Type 40: Low Heat of Hydration. This cement is used where the rate and amount of heat generated must be minimized. It develops strength at a slower rate than Type 10 cement.

e) Type 50: Sulphate Resistant. This is used only in areas subject to severe sulphate action. It gains strength more slowly than Type 10 cement.

The weight ratio of water to cement by weight is important for the quality of a cement grout. Excess water results in bleeding of the mix (water segregation), lower strength, greater shrinkage and lower durability. Relationships between water/cement ratios to unconfined compressive strength is shown in Figure 4 (Littlejohn, 1976). The strength of the grout is also a function of time as shown in Figure 5. Usually a compressive strength of 30MPa can be developed within seven days for a good quality grout. This is considered to be a minimum for purposes of design (Hubst, 1983). The most suitable water/cement ratios for grouts is 0.4 - 0.45. The grout is still sufficiently fluid for pumping at these ratios exhibiting minimal shrinkage and high strength.

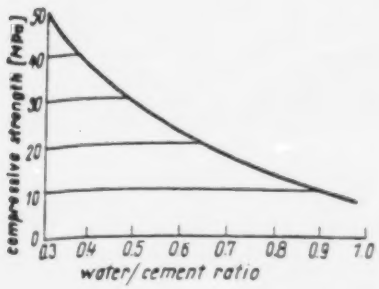


Figure 4: Effect of water content on the compressive strength of grout. (Littlejohn, 1976).

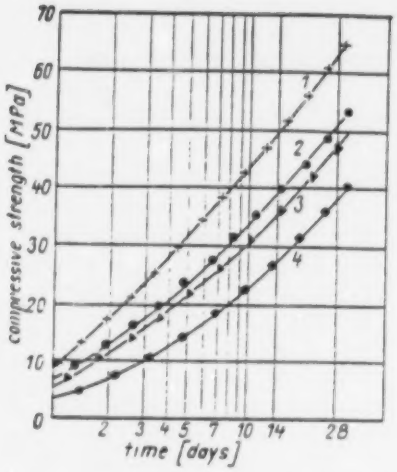


Figure 5: Gain in strength with time of ordinary Portland grouts at various water/cement ratios:
1 - 0.4, 2 - 0.45
3 - 0.5, 4 - 0.60
(Littlejohn, 1976)

2.2.1 Additives

Table 1 summarizes a list of grout additives that are available to accelerate or retard setting time, prevent shrinkage and induce expansion and prevent bleeding.

A combination of several types of additives with intention of obtaining a combined effect is not recommended (Hubst, 1983). Limited knowledge exists on the use of grout additives and quality control is extremely important. Figure 6 summarizes the above observations for pull-out tests performed at the Strathcona Mine (Bharti, 1983).

Table 1

GROUT ADDITIVES (LITTLEJOHN, 1976)

Additive	Active Chemical	Amount (% of cement by weight)	Remarks
Accelerator	Calcium Chloride	1-2%	Accelerates setting and hardening
Retarder	Calcium Lignosulphonate	0.1-0.5%	Retards setting and increases fluidity
Expander	Aluminum Powder	0.005-0.02%	Up to 15% expansion
Anti-bleed	Cellulose Ether	0.2-0.3%	Equivalent to 0.5% of mixing water
Fluidifier	Bentonite	2-3%	Acts as anti-bleed

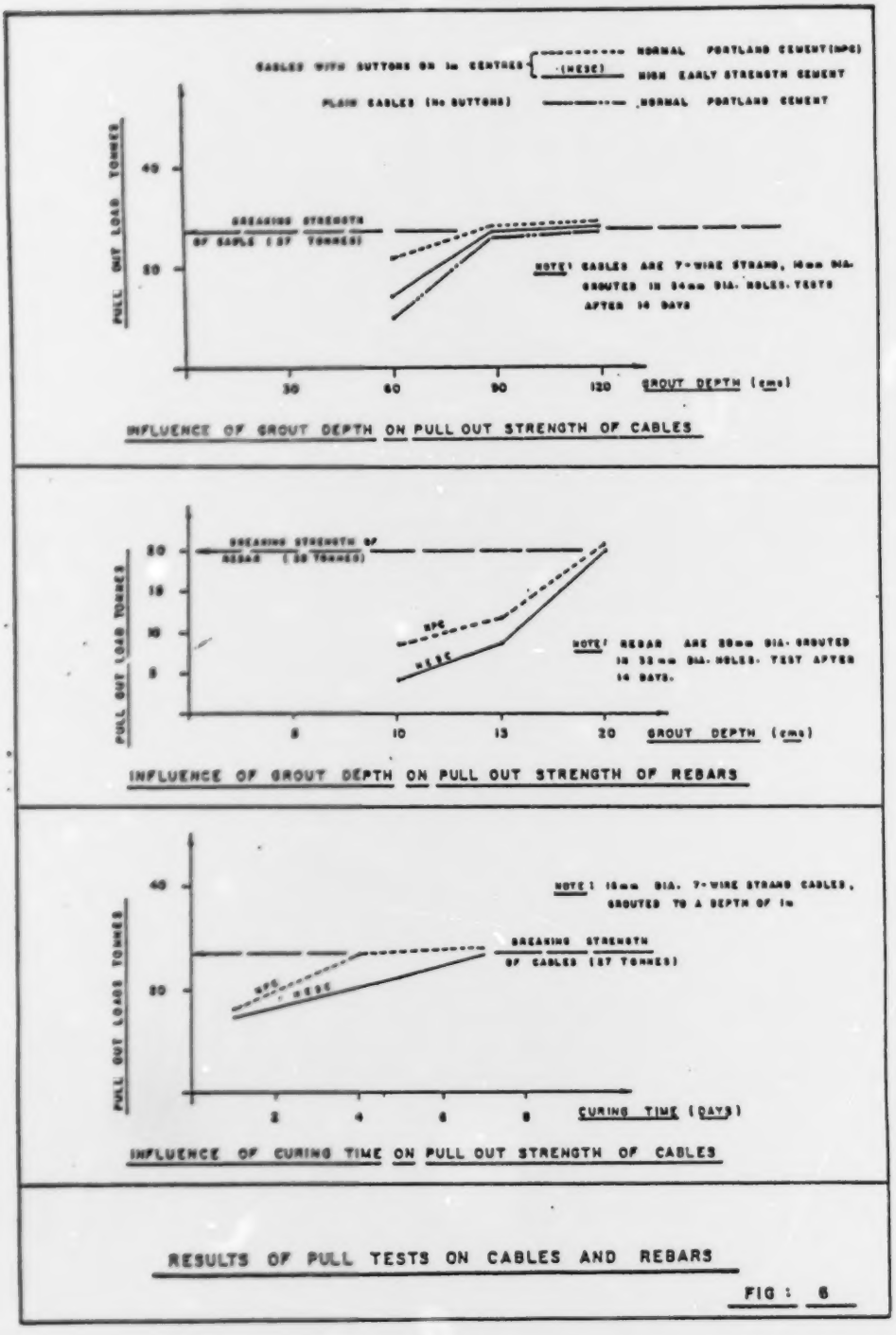

CABLES WITH BUTTONS ON 1m CENTRES
NORMAL PORTLAND CEMENT (NPC)
(NEAC)
HIGH EARLY STRENGTH CEMENT
PLAIN CABLES (No BUTTONS)
NORMAL PORTLAND CEMENT
PULL OUT LOAD TONNES
BREAKING STRENGTH OF CABLE (57 TONNES)
NOTE: CABLES ARE 7-WIRE STRAND, 16mm DIA. GROUTED IN 64mm DIA. HOLES. TESTS AFTER 14 DAYS
GROUT DEPTH (cms)
INFLUENCE OF GROUT DEPTH ON PULL OUT STRENGTH OF CABLES
BREAKING STRENGTH OF REBAR (20 TONNES)
NOTE: REBAR ARE 20mm DIA. GROUTED IN 32mm DIA. HOLES. TEST AFTER 14 DAYS.
INFLUENCE OF GROUT DEPTH ON PULL OUT STRENGTH OF REBARS
NOTE: 16mm DIA. 7-WIRE STRAND CABLES, GROUTED TO A DEPTH OF 1m
BREAKING STRENGTH OF CABLES (57 TONNES)
CURING TIME (DAYS)
INFLUENCE OF CURING TIME ON PULL OUT STRENGTH OF CABLES
RESULTS OF PULL TESTS ON CABLES AND REBARS
FIG : 6

Figure 6: Pull-out tests on cables and rebar at the Strathcona Mine. (Bharti, 1983)

2.3 OTHER METHODS

Farah (1986) has shown that the use of concrete rather than the use of straight Portland cement will have a higher ultimate bond strength and a higher overall ductility. This is shown in Figure 7. Jeremic (1983) has shown that a steel button or anchor placed at the end of a cable may increase the bond strength two-fold. This has been established through testing whereby for a cement/water ratio of 0.4, a bond strength of 2.8MPa was developed without the use of the button and 6.2MPa with a button. This is most likely due to the redistribution of stress at the base of the button as shown in Figure 3b. The grout must fail largely by compression in the vicinity of the button. Grout is ten times as strong in compression as it is in tension.

In addition, Lappalainen (1983) has shown that a clean steel cable with a fibre core has four times as high a bond strength as that exhibited by a greased or dirty cable.

The grout/cable bond is largely dependent upon the area in contact as shown by equation 1. The objective is to increase the surface area to grout contact, Figure 3c, thereby enabling a higher shear strength to be developed. In addition, Lappalainen (1983) has shown that a high residual strength can be achieved after a cable loses its peak bond strength. This is due to friction caused by the cable/grout/rock interface. This is enhanced by increasing the roughness and irregularity of the cable surface. Fuller (1978) found that a layer of surface rust and surface indentations in the wire substantially increased the bond strength, Figure 8. Cox (1978) has shown that debonding at the grout-wire interface does not commence until the tensile strain in the wire exceeds 8% of its yield strain.

Stillborg (1984) has shown through pull-out tests that if reinforced rock is subjected to dynamic loading from a nearby production blast, the load bearing capacity of the cable will only be affected when the particle velocity exceeds 500mm/s at the cable.

The hole size relative to the size of cable employed is generally between .13 to .19 (Cable area/hole area) as per BBR specifications. Schmuck (1979) suggests that 6 to 13mm of grout exist between the hole and the cable to ensure maximum grout bond. Table 2 shows the range of hole/cable sizes employed by various mine operators. Generally 54mm diameter holes are employed when one 15.2mm cable is installed and 64mm diameter holes are employed when using two 15.2mm cables.

It is generally concluded that fully grouted cables only require a bond length between 16 to 30 x cable diameter in order to resist pull-out. A water/cement ratio of under .45 by weight will result in an acceptable shear strength employing a seven day curing period. A good quality grout will negate the use of employing additives to improve bond strength. However, by reducing the required bond length through the use of buttons, concrete etc. one may increase the ability of supporting layered blocks which

have a small bond length to failure volume and results in the block to be pulled off the cable due to its weight.

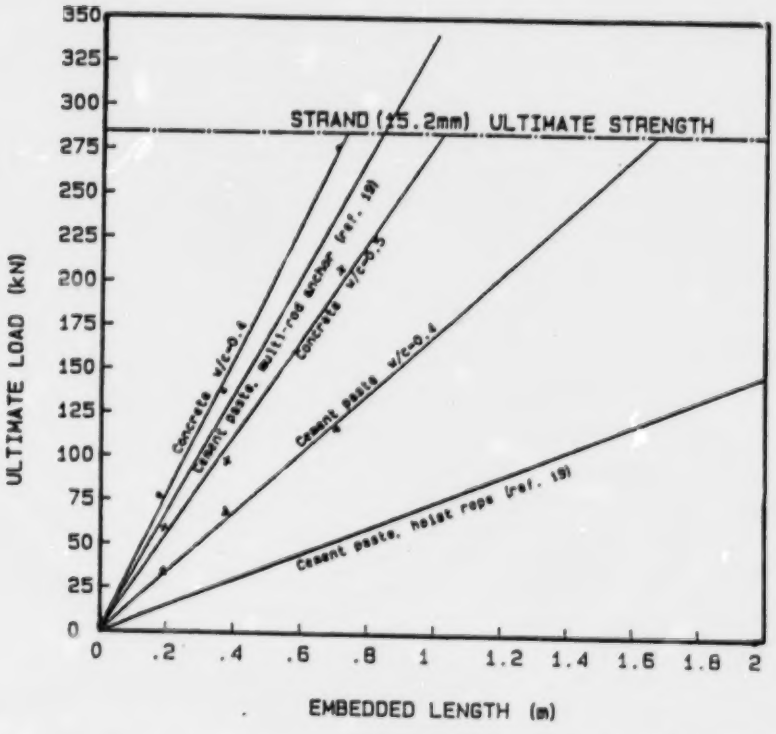


Figure 7: Employing concrete as a grout alternative.
(Farah, 1986)

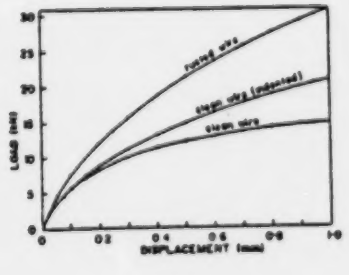


Figure 8: Pull-out test results on single 7mm diam. high tensile steel wires with different surface conditions.
(Fuller, 1983)

Table 2: SURVEY OF CABLE BOLT PRACTICES

MINE	APPLICATION	BOLT LENGTH	PATTERN	HOLE SIZE	GROUT	BOLT SIZE/TYPE	YIELD LOAD	OTHER
1 Brunswick Mining and Smelting Brunswick #12	Cut & Fill 4.9m lifts	17.7m	1.8m x 1.8m (upholes)	57mm(2-1/4")	high early cement(HE) expansive additive water/cement = 0.4	2 x 16mm (5/8") strand	53 tonne	6mm (1/4") I.D. breather tube
2 Falconbridge Strathcona	Cut & Fill 3m lifts, 3m overlap areas with RQD<50%	17m	1.8m x 1.8m	54mm(2.1")	portland cement-sand water/cement = 0.4 cable bunsens used	1 x 16mm strand	27 tonne	6mm breather tube 12.5mm (1/2") grout tube
3 INCO Thompson	Cut & Fill	6.1m, 12.2m	1.2m x 1.2m(1 cable) 2.4m x 2.4m(2 cables)	54mm	portland cement-sand water/cement = 0.45	1 or 2 x 16mm strand	27 or 54 tonne	9.5mm(3/8") breather with Mono pump 12.5mm breather with Spadel pump
4 Con Mine Yellowknife	Cut & Fill 3m lifts + 4.6m H/W	10m, 23m	2.2m x 2.4m	57mm	portland cement-sand water/cement = 0.33	2 x 16mm strand	58 tonne	6mm breather tube
5 New Broken Hill Australia	Cut & Fill 2.4m lifts	20m	2m x 2m	57 mm	high early cement bleeding additive water/cement = 0.51	2 x 16mm strand	53 tonne	3mm breather tube
6 Homestake S. Dakota	Open Stope H/W support	15m	3m x 3m	57 mm	portland cement-sand water/cement = 0.5	1 x 16mm strand	29 tonne	bunsens used
	Cut & Fill	18.3m	1.8m x 1.8m	64 mm	portland cement-sand water/cement = 0.5	2 x 16mm strand	58 tonne	12.5mm breather tube bunsens used
7 LynnGold MacLellan	Open Stope H/W & F/W	6m	2.4m spacing (2 holes) every 20m sub-level	54 mm	portland cement-sand water/cement = 0.45	2 x 16mm strand	58 tonne	holes drilled from sub-levels at wall contact
8 Placer Dome Campbell	Open Stope H/W support	15m	2.4m spacing (3 holes)	51 mm	portland cement-sand/HE water/cement = 0.4	1 x 16mm strand	29 tonne	holes drilled from sub-levels at wall contact
	Cut & Fill 3m lifts	15m	2.4m x 2.4m	51 mm	portland cement-sand/HE water/cement = 0.4	1 x 16mm strand	29 tonne	support dead weight equivalent is 0.2 m exposed width(F/W - H/W
9 West Coast Mines Tasmania	Open Stope H/W support	20m	1.8m x 1.8m (perpendicular to H/W)		portland cement-sand water/cement = 0.5	7 x 7mm wire	45 tonne	holes drilled from sub-levels breather tube 12.5mm, 19m grout tube bunsens used
10 Outokumpu Oy Finland	Open Stope H/W support	30m to 15m	1 bolt per 25 sq. meters to to 1 bolt per 50 sq. meters	41mm	portland cement-sand water/cement = 0.5, 0.45	2 x 15mm strand	50 tonne	Fan shaped from F/W and H/W access with spacing 3m to 10m
				64mm	portland cement-sand water/cement = 0.5,0.3	1 x 15 mm strand	25 tonne	Fan shaped from F/W and H/W access with spacing 3m to 10m
11 Mamberget Mines Sweden	Open Stope F/W support	25m to 15m	2.5m along strike (1 hole) every sub-level (20m vertical)		portland cement-sand water/cement = 0.5, 0.45	1 x 36 mm strand	90 tonne	drilled at F/W contact from sub-level RQD of F/W 20 to 60%
12 Mount Isa Mines Ltd. Australia	Open Stope H/W support	9-12m	2m spacing (3 holes) every sub-level (30m vertical)	57-70 mm	portland cement-sand water/cement = 0.5	2 x 15mm strand	50 tonne	holes drilled from sub-levels at H/W contact, first 2m debonded
13 Wenmin H-W Mine	Cut & Fill	15m Open Stope	2m x 1.5m	51mm	portland cement-sand	2 x 15mm strand	50 tonne	
14 Eldorado Beaver Lodge	Cut & Fill	15m	1.8m x 2.4m	57mm	portland cement-sand bleeding additive	1 x 13mm strand		Spadel grout pump employed

3.0 INSTALLATION

This section outlines the proper method of installing cables in order to maximize the support potential.

3.1 PATTERN

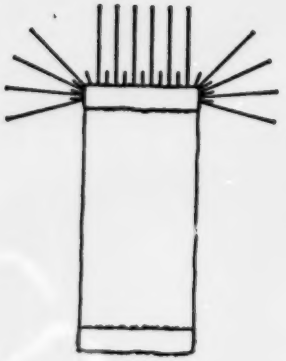
The layout of a cable bolt pattern in an open stope environment is largely restricted to access. Most open stopes are planned with minimal development and therefore it is not possible to place an even distribution of cables such as would be possible with cut and fill. Figure 9 shows various configurations and applications of cables for open stope support. It is generally agreed by most researchers that the potential modes of failure be identified and the support be implemented to increase the overall factor of safety. This is shown by Figure 10 whereby a Factor of Safety of 1.5 to 3 is recommended (Stillborg,1986). Design, however, is largely empirical, Figure 11, and is a function of stress environment and rock conditions. This is due to the difficulty of delineating/predicting the occurrence of individual failure modes prior to mining and how they will influence the stope.

Generally no accepted guidelines exist for the design of cable bolt systems and rules of thumb that have been developed for mechanical bolts are still applied. Table 2 summarizes the cable bolt practices of 14 mine operators. The information has been compiled through literature review and personal communication. The table attempts to summarize practices in Canada, U.S.A, Australia, Sweden and Finland.

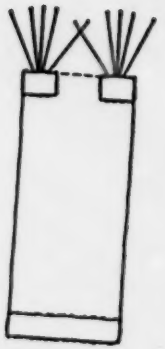
Potvin (1988) in his study of open stope operators found that among 34 Canadian mines visited during the study (1986-1988) that twenty mines used cable bolts with a rate of success of 75%, Table 3. He found that a minimum bolting density used in stope backs is 0.1 cables per square meter (ie. 3m x 3m pattern) and cable bolts were not effective for RQD/Jn/Hyd.Rad values less than 0.75. RQD refers to the rock quality designation, Jn to the joint number (Barton) and the HYd.Rad refers to the hydraulic radius which is the surface area exposed divided by the perimeter. This relationship is shown in Figure 12 and is empirically derived. The optimum bolt length was equivalent to the span exposed. The data base consisted of 68 observations, 59 of which were backs. Generally patterns ranged from 3.0m x 3.0m to a minimum of 1.4m x 1.4m. The average pattern was 1.8m x 1.8m.

Cable bolt design is largely derived from cut and fill practice. Fuller (1981) had instrumented a cut and fill stope back employing a 4m x 4m and 2m x 2m pattern at the New Broken Hill Consolidated mine in Australia. He found through instrumentation that the zone of tension was restricted to within 2m of the back. He concluded that the cables allowed the remaining back to reach a self-stabilizing, arched

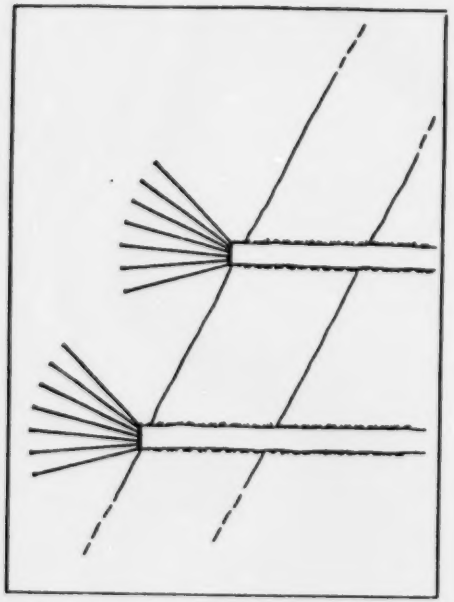
Uniform cable bolt pattern installed in open stope overcute.



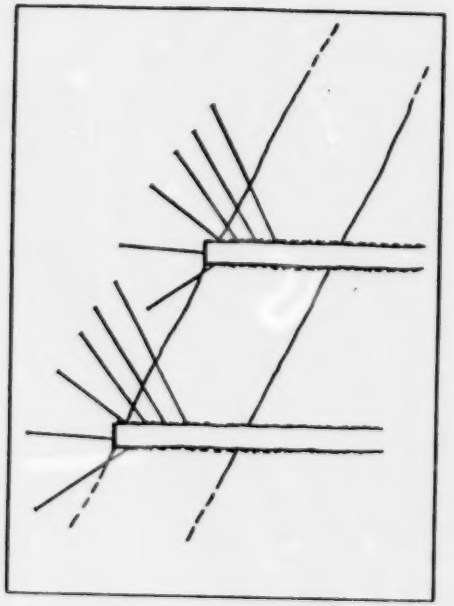
Uniform cable bolt pattern installed in open stope overcute and supplemented with short rebar.



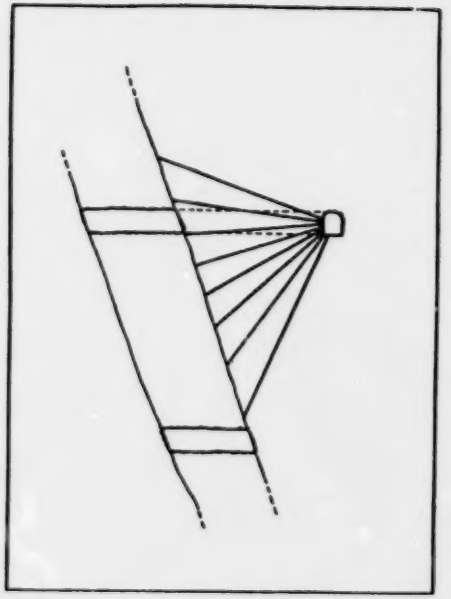
Cable bolt support system designed for overcute containing a small pillar(s).



Creation of a rock bolt in the hanging wall by installing a localized high density of cable bolts.

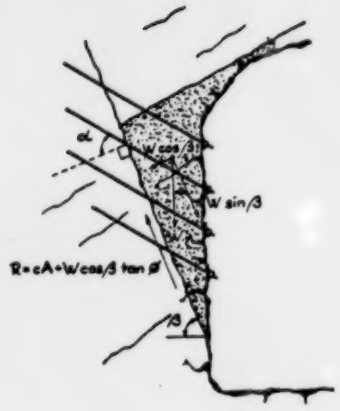


Uniform cable bolt pattern installed in an open stope wall.



Cable bolt support system for a hanging wall, installed from a parallel bolting drift.

Figure 9: Cable Bolt Application - Open Stope



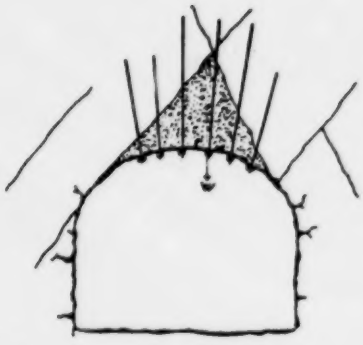
We have

$$N = \frac{W(f \sin\beta - \cos\beta \tan\phi) - cA}{B(\cos\alpha \tan\phi + f \sin\alpha)} \qquad (6)$$

where

N = number of rockbolts
W = weight of wedge including the weight of any rock that is supported by the wedge, (density of rock approximately 2.7 t/m³)
f = safety factor. It is often found that $1.5 \leq f \leq 3$. The value of f depends on the damage which would result from sliding of the wedge and also on whether permanent or temporary reinforcement is to be considered
β = dip of the sliding surface
ϕ = angle of friction of the sliding surface
c = cohesive strength of the sliding surface
A = base area of the sliding surface
B = load bearing capacity of the bolt, see also section 5.1
α = angle between the plunge of the bolt and the normal to the sliding surface

Reinforcement of a wedge of rock free to slide under its own weight.



We have

$$N = \frac{W \times f}{B}$$

where

N = number of rockbolts
W = weight of wedge, (density of rock approximately 2.7 t/m³)
f = safety factor, usually $2 \leq f \leq 5$
B = load bearing capacity of the bolt, see also section 5.1

Reinforcement of a wedge of rock free to fall under its own weight.

Figure 10: Cable pattern design - analytical (Stilborg,1986)

Parameter	*Empirical rule*
Minimum length and maximum spacing	
Minimum length	Greatest of: (a) 2 x bolt spacing (b) 3 x thickness of critical and potentially unstable rock blocks (Note 1) (c) For elements above the springline: spans <6 m: 0.5 x span spans between 18 and 30 m: 0.25 x span spans between 6 and 18 m: interpolate between 3 and 4.5 m (d) For elements below the springline: height <18 m: as (c) above height >18 m: 0.2 x height
Maximum spacing	Least of: (a) 0.5 x bolt length (b) 1.5 x width of critical and potentially unstable rock blocks (Note 1) (c) 2.0 m (Note 2)

Notes:

1. Where joint spacing is close and span relatively large, the superposition of two reinforcement patterns may be appropriate (e.g. long heavy elements on wide centres to support the span, and shorter, lighter bolts on closer centres to stabilise the surface against ravelling).
2. Greater spacing than 2.0 m makes attachment of surface support elements (e.g. weldmesh or chain link mesh) difficult.

Figure 11: Cable pattern design - empirical (US Army Corps of Engineers,1980)

Table 3: Open Stope Operators Survey (Potvin, 1988)

- Algoma Steel Corp. Limited - G. W. Macleod Mine
- ■ Barrick Resources - Camflo Mine
- ■ BP Canada Inc. - Mines Selbaie
- Cambior - Niobec Mine
- Corporation Falconbridge Copper
  - ■ Corbet Mine
  - Lac Shortt Mine
- Dome Mines Limited
- Falconbridge Limited
  - ■ East Mine
  - ■ Fraser Mine
  - ■ Kidd Creek Mine
  - ■ Lockerby Mine
  - Mining Technology Division
  - ■ Onaping Mine
  - ■ Strathcona Mine
- Hudson Bay Mining and Smelting
  - Centennial Mine
  - Chisel Lake Mine
  - Flin Flon Mine
  - Spruce Point Mine
- Inco Limited
  - Little Stobie Mine
  - Mine Research Division
  - ■ Stobie Mine
  - ■ Thompson Division
- ■ Kiena Gold Mines
- Noranda Minerals Inc.
  - ■ Brunswick Mining and Smelting
  - ■ Chadbourne Mine
  - ■ Geco Mine
  - ■ Golden Giant Mine
  - Lyon Lake Mine
  - Mattabi Mine
  - Mattagami Lake Mine
  - ■ Mines Gaspe
  - Mining Technology Division
  - ■ Norita Mine
- Pamour Porcupine Mines Limited
  - ■ Ross Mine
  - No. 1 Mine
- Sherritt Gordon - Ruttan Mine
- ■ Westmin Resources Limited

■- Cable bolts employed as support

condition soon after blasting. Thereby patterns of 2m and 4m square were employed with between 1 and 2m of overlap between lifts.

Empirical methods have been employed by Hunt (1977) in the support design at the Zinc Corporation and New Broken Hill Consolidated Ltd. operations in Australia. They are based upon design principles devised by the Snowy Mountain project in Australia (Lang,1961) and are summarized in Figure 11. Cable bolt pattern and design at these operations are based on supporting the dead weight resulting from a 2.4m lift and 70% of the ultimate tensile bolt strength. This results in a 2m x 2m pattern.

Strathcona (Bharti, 1983) employs a 1.8m x 1.8m pattern in the back of their cut and fill stopes. Based on past experience it was observed that the height of failure is limited to 0.2 to 0.25 x stope width and is in the form of wedges. Therefore for a 13m panel the zone of instability is limited to 3m above the back which is equivalent to one lift height. Cables are designed to support this load based on dead load. An overlap of at least one cut (3m) is provided when a stope has to be re-cabled. This ensures a design anchor length of 3m beyond the potential zone that is to be supported. Typical joint spacing is 1m to 3m with an RQD of 40-50%.

Lappalainen (1983) through past practice at Outokumpu Oy open stope mine operations has shown that a cable bolt spacing of one per 25 square meters to one per 50 square meters of exposed wall will be effective. This was found to be sufficient when using fully grouted steel strands (2 x 15.2mm/hole) or steel cables (25 to 35mm diam.). Cable bolt lengths varied between 15m to 50m and were generally fan shaped, Figure 13. It was shown that the placement of cable support at the Kotalahti Mine resulted in a recovery of 100% and a dilution of 18%. This was compared to an adjacent unsupported stope whereby 67% recovery and 45% dilution was realized. Design is largely empirical due to the many variables involved ie. fracture density, friction between blocks, access.

Consequently the pattern of cable bolts for open stope wall support have evolved from the practices observed in cut and fill operations. However, differences exist in the limited access and stress conditions associated with large exposed spans common to open stope operations. Design is largely empirical and is developing through the accumulation of practical experience. Patterns along stope walls are generally widely spaced, however, the practice of strapping the collars between cables has shown success in terms of achieving lower dilutions (Bywater (1983), Figure 14. Australian open stope practice (Fuller, 1983) is to space cable bolt rings 1.5 to 3m apart along the strike of the stope.

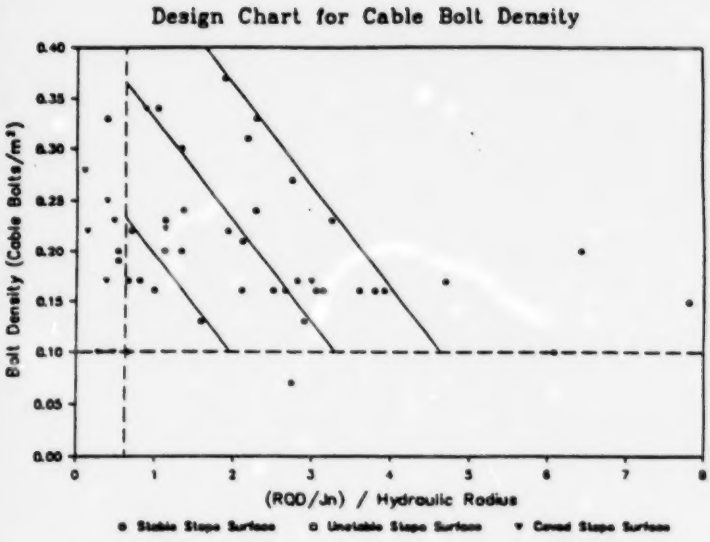
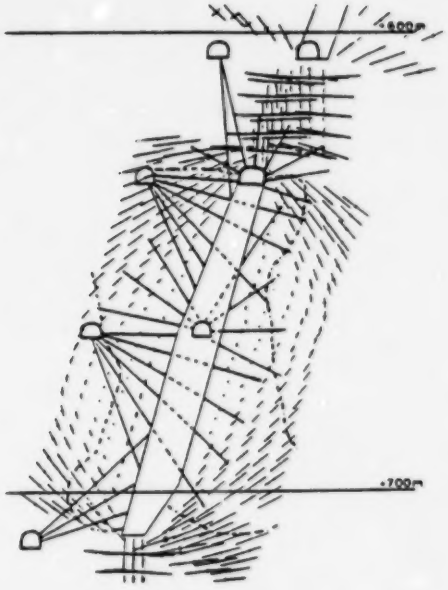


Figure 12: Cable Bolt Design Chart (Potvin, 1988)



BEM calculation and cable bolting plan. Tensional areas contoured by dotted lines. Bolting mainly from the decline on the hanging wall side ; Orebody sections left without cables.

Figure 13: Outokumpu Oy cable bolt pattern

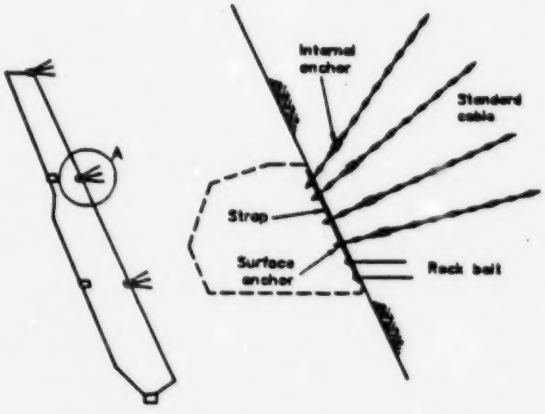


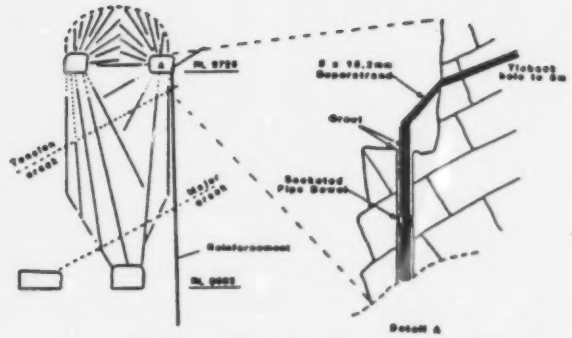
Figure 14: Reinforcement of a hanging wall with cables and surface strapping.

It is generally accepted that the formation of a loose zone and/or stope closure is inevitable. Lappalainen (1983) has found that fully grouted cables can strain up to 2-5% before breaking. This produces a stabilizing compressive normal force across the fractures. Lappalainen found that even though the cable bolt density is not high enough to prevent fracturing it does inhibit caving. The supported rock acts as a self-supporting structure.

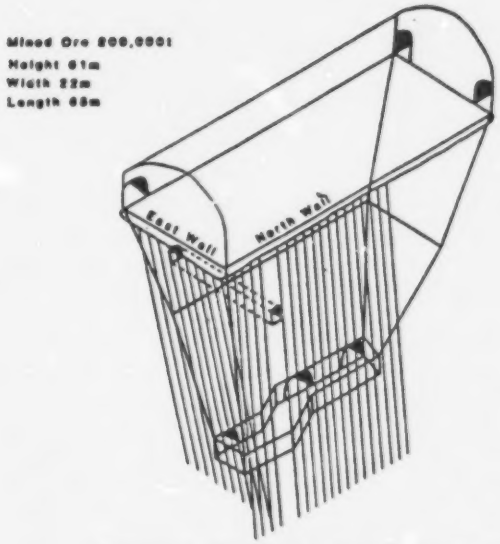
Mandolin bolting (Thompson, 1987) has been employed in blocky ground in the walls of an open stope as shown in Figure 15. This consisted of 114mm diam. steel pipes (socketed) installed in bore holes drilled parallel to the stope walls. In addition three high tension strength steel cables were installed untensioned down the centre of the pipe in order to provide tensile strength across the pipe joins. The entire assembly was grouted and the top of the cables anchored to the stope back/walls. Instrumentation of the dowels confirmed loading in tension and compression. Overbreak was significant within the unreinforced areas however was limited in the supported areas. This type of pattern support has application to longitudinal pillars within blocky ground which are considered to be in a relaxed state.

Thompson (1987) had found that load transfer is improved locally by use of anchors as shown in Figure 14 as well as a barrel and wedge surface anchor in order to secure straps between bolts. Bywater (1983) had debonded the first 2m of cable at the stope wall contact at Mount Isa mines in Australia. This was based on previous observations whereby it was observed that surface movements of 50mm were possible whereas only 20mm was sufficient to break the cable. Surface straps between cables and anchors were mechanically attached with barrel and wedge anchors, Figure 16. These were necessary due to the thinly bedded layers that would adhere to the cable. This resulted in a low bond length for each block and hence the support force developed was small compared to the block weight. This resulted in a successful application in the mining of a layered shale hanging wall.

The stability of crown pillars subjected to mining induced stresses is discussed by Mathews (1983). Debonding of the centre portion of the pillar was practised in order to ensure that the pillars would yield and still remain intact. A series of barrel and fittings were applied along the length of the cable in order to increase the load transfer between the grout and the cable by attempting to compress the grout in front of the fitting rather than by shearing at the strand-grout interface, Figure 3. Thompson (1987) has shown that the required embedment length can be reduced to 25 cm. with the use of barrels/anchors along the length of the grouted cable.

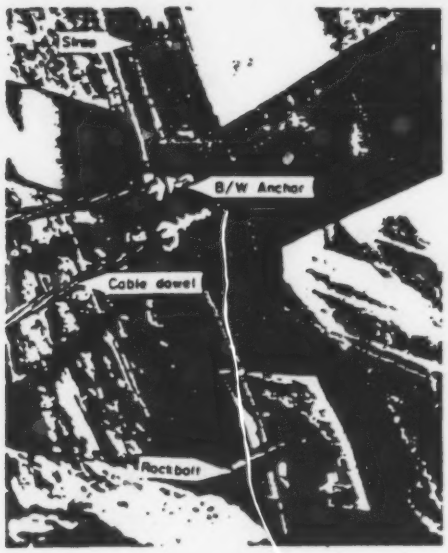


Cross section of stope showing high shear strength dowel reinforcement.



Isometric view of the stope showing areas of reinforced wall.

Figure 15: Mandolin Bolting - Application



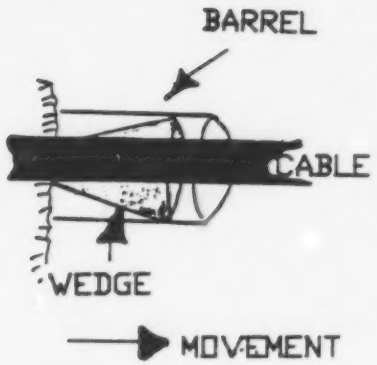


Figure 16: Anchoring of cables to surface of hanging wall.

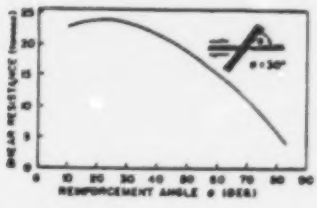
Developing a pattern either in the back or the hanging wall of an open stope is largely empirical and site specific. The stiffness of a cable bolt system which is a function of bolt density and bond length of cable should be matched to the function for which it is designed. If too stiff, the rock mass will be restrained from deforming and the stored energy may result in sudden failure of the support system. A low stiffness may result in too much movement and as a consequence reduce the stabilizing effect significantly (Stheeman, 1982). The debonding technique has been employed in Australia but has not been put into practice in Canadian operations.

In order to employ these concepts one must understand how a supported rock mass would behave upon reinforcement. This requires knowledge of the rock mass strength and the load transfer properties between the jointed rock mass and the grouted cable. In order to achieve a proper anchor length it is suggested that bond lengths defined in section 2 be employed for purposes of assessing cable length.

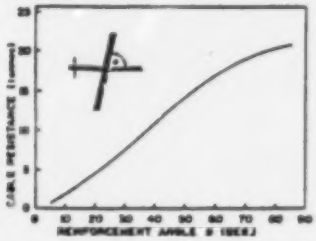
3.2 INCLINATION OF CABLES

Apart from bolt density and length of cable, the inclination of the drill hole has the greatest effect on being able to utilize the maximum cable bolt strength. Fuller (1983) has shown, through pull-out tests, that for a cable to resist shear movement it should be installed between 15 and 30 degrees to the joint surface and be placed in tension by the shear movement. Joint opening is best restricted by placing cables normal to the joint plane. This is shown in Figure 15. This assumes that the friction angle is approximately 30 degrees. Miller (1983) further evaluated the shear effect in Figure 18 whereby a cable oriented at 90 degrees to the shear movement will result in failure by pure shear and must rely on the shear strength of the cable. This resulted in a reduction in strength by 80%.

Barton (1983) through laboratory tests has related the inclination of a bolt to the plane of shear once movement has occurred. He showed that fully grouted bolts installed before any shear stresses develop, will cause the bolts to be subjected to full excavation induced shear stress, with little if any assistance from the inherent strength of the joints (friction angle and cohesion). The ideal situation is one where the shear strength of the natural joint is mobilized through only a few millimetres of movement and would add to the overall strength of the support system. This requires one to debond a portion of a cable in order to utilize the available strength of the joint upon displacement. This would require that cables be anchored only at the ends and thereby considerable extension would be possible in the central ungrouted region. The mobilized angle of friction would thereby be increased due to dilation along the joint surface.

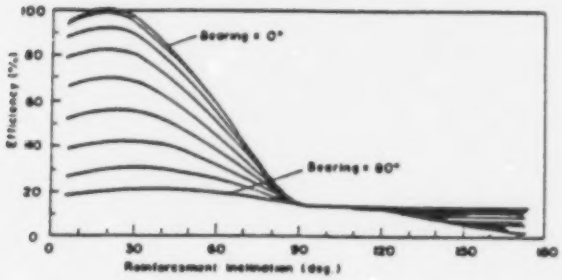


a) resistance to joint shear - shear displacement 1.8mm



b) resistance to joint opening - joint opening 1.8mm

Figure 17: Effect of cable inclination on cable resistance on one cable. (Fuller, 1983)



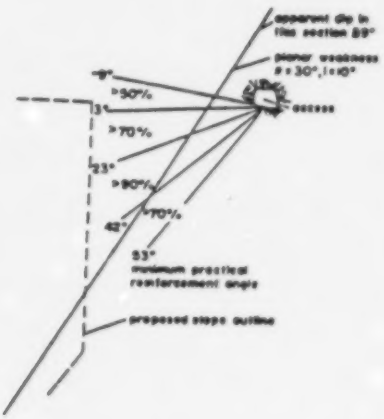


Figure 18: Cable efficiencies for a range of inclinations. A bearing of 0° implies trend of bolt is perpendicular to strike of structure. (Miller, 1983)

3.3 PROCEDURES

It is critical that the cable is clean and that the grout mixture is less than 0.45 water/cement by weight (preferably 0.3 to 0.4).

3.3.1 Downholes

The hole is cleaned first and pumped dry. The grout hose is placed at the bottom of the borehole and retrieved as the hole is filled with cement. The cable is installed prior to grouting (Australian practice) or as suggested by Stillborg (1986) the cable is pushed down the hole through the grout.

3.3.2 Upholes

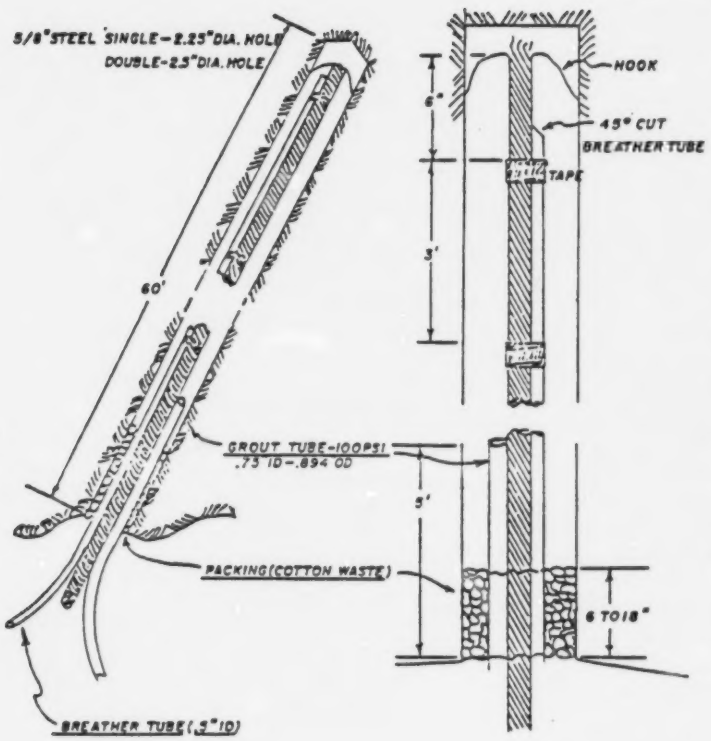
The following procedure should be used in horizontal to upward inclined holes.

1) Cable is inserted to the bottom of borehole together with a stiff nylon or polyethylene breather tube. The breather tube (6mm to 10mm) should be located approximately .3m from the end of the cable. It should be tapered so as not to become plugged, Figure 19. Spring clips/anchors should be installed at the end of the cable in order to secure it to the end of the hole. The hole can be cleaned by flushing the hole via the breather tube (Schmuck,1983).

2) The grout tube (19mm) is inserted 0.5m into the hole. The hole collar is sealed by employing fabrene, steel wool etc.

3) The grout is pumped into the hole. The breather tube should be immersed into a bucket of water and the displaced air monitored. When the air bubbles stop or grout appears in the air tube then the breather and grout tube are sealed and cut. The size of breather tube is dictated by the type of grout pump employed. A 13mm (1/2") diameter breather tube is employed with a Spedel grout pump and a 10mm (3/8") diameter tube for a Moyno pump.

Cables can be manually pushed up the hole by attaching the cable to the drill rig feed mechanism (Schmuck,1983) or through the use of fully mechanized cable bolting rigs.



1. Drill $2\frac{1}{2}$ or $2\frac{1}{2}$ inch dia. (as required) hole to specified length.

NOTE

$2\frac{1}{2}$" dia. for single & $2\frac{1}{2}$" dia. for double cablebolts.

Upholes

2. Bend one or two wires back on end of cable to form hook. Cut breather tube end at 45° angle and tape to cable 6" & 3' below hook.
3. Insert cable in hole using drill rig centralizer and chain cable grab. Insure that cable grab goes only around cable and does not crush breather tube. Push cable in hole brushing off "scaly' rust as inserted.
4. When in place, flush breather tube with water to clear hole and wash walls.
5. Push grout tube 5' into hole and pack collar (at least 6" depth) with cotton waste to seal
6. Pump mixture (2 parts Portland Cement- 1 part water) into grout tube with pressure pot until mixture bleeds out breather tube.
7. Tie off grout and breather tube 1 foot from collar.

Downholes

8. Tape grout tube loosely to end of cable and lower into hole. No hook or breather tube is required for downholes.
9. Wash tube with a small amount of water to lubricate then pump grout while pulling tub out of hole. When grout issues from the to of the hole pull the grout tube out of the hole.

Figure 19: Cable bolt installation (Schmuck, 1979)

3.4 COST OF INSTALLATION

A breakdown of cable bolting costs (1988) for the Ruttan operation to install two 16mm (5/8 inch) cables within a 51mm diameter uphole is as follows:

- 2 x 16mm diam. cable (29tonnes each)	= 2 x $ 2.15/meter
- grout(19mm) + breather (10mm) tube	= 0.56/meter
- Grout pump rental	= 390.00/month
- Mixer rental	= 415.00/month
- Portland cement	= 12.00/25kg bag

Cost per meter of drill hole (51mm diam.)

- cable		= $4.30/meter
- cement		= $3.48/meter
- pump/mixer/grout tube		= $3.28/meter
- labour	- drilling	= $14.50/meter
	- installation	= $3.28/meter
TOTAL		$28.84/meter

The Detour Lake operation (cut and fill) in Timmins, Ontario has an installed cost of $21.00/meter (1 x 15.2mm) whereas the MacLellan operation (open stope) in Lynn Lake, Manitoba has an installed cost of 20.00/meter (2 x 15.2mm). These costs included materials and labour.

4.0 CABLE SPECIFICATIONS

The type of cables in use can be summarized as follows:

a) 7 x 7mm diameter high strength steel wire dowels (approximately 46 tonne minimum breaking strength);
b) single 15.2mm diameter strand cable (7 wire) (approximately 26 tonne minimum breaking strength);
c) birdcage cables (35mm,46mm diameter) (25 tonne, 50 tonne minimum breaking strength).

Table 4 (BBR specifications) summarizes typical cable bolts employed by the mining industry. The most common cable employed in the mining industry for cable reinforcement is the 15.2mm (7-wire) strand, normally installed in units of two. A more recent development (Stillborg,1986) is the "Flexirope" whereby a 28mm diameter stranded cable will develop a 50 tonne yield load.

Table 4: BBR Wire Rope Specifications

TYPICAL CABLEBOLTS
FOR UNDERGROUND MINING

MINIMUM BREAKING LOAD	KN	196	262	393	458	933	869	785	1046
	Tons 1000	43	58	78	88	113	128	188	225
NOMINAL MASS	Kg/m	0.9	1.2	1.8	2.1	2.4	2.7	3.6	4.8
	lb/ft	0.6	0.8	1.2	1.4	1.6	1.8	2.4	3.2
NOMINAL HOLE DIA (mm)	Unions	35	35	50	50	50	50	64	64
	Tons	35	35	50	50	64	64	64	75
WIRE TYPE	Ø 7	3	4	6	7	8	9	12	15
STRAND TYPE	Ø12.5	1	–	2	–	3	3	4	5
	Ø15.2	–	1	–	2	2	–	3	4

STEEL GRADE	High Tensile – 1700 to 1850 MPa (235 to 270,000 p.s.i.).
ELONGATION	3.5% on 600 mm gauge length immediately prior to fracture.
PROOF STRESS	Minimum 80% of breaking load at 1% elongation.
RELAXATION	Maximum 7% from initial load at 70% of breaking load after 1000 hours at 20°C.

A cable which will result in improved bond/cable strength and therefore smaller embedment lengths is the "birdcage type" as shown in Figure 20. This cable results in a stiff system due to the mechanical interaction of the non-uniform cross- section and increased steel/grout interface (eqn. 1). Grout can fully encompass the individual cables resulting in a more competent grout/steel bond. It was found that less than 1m of embedment length was found necessary (Hutchins,1986) to cause rupture of the birdcage in pull tests. Capacities of up to 50 tonnes are possible. Rock Engineering Pty. (1986) states that the pull-out strength is double that of cementitious grouts due to crushing of grout inside and outside the birdcage rather than relying on shear load failure. The pull-out is less likely to be affected by surface coatings on the individual wires. A 50 tonne birdcage is 46mm in diameter and requires a 57mm diameter drill hole whereas a 25 tonne birdcage is 35mm in diameter and requires a 64mm drill hole.

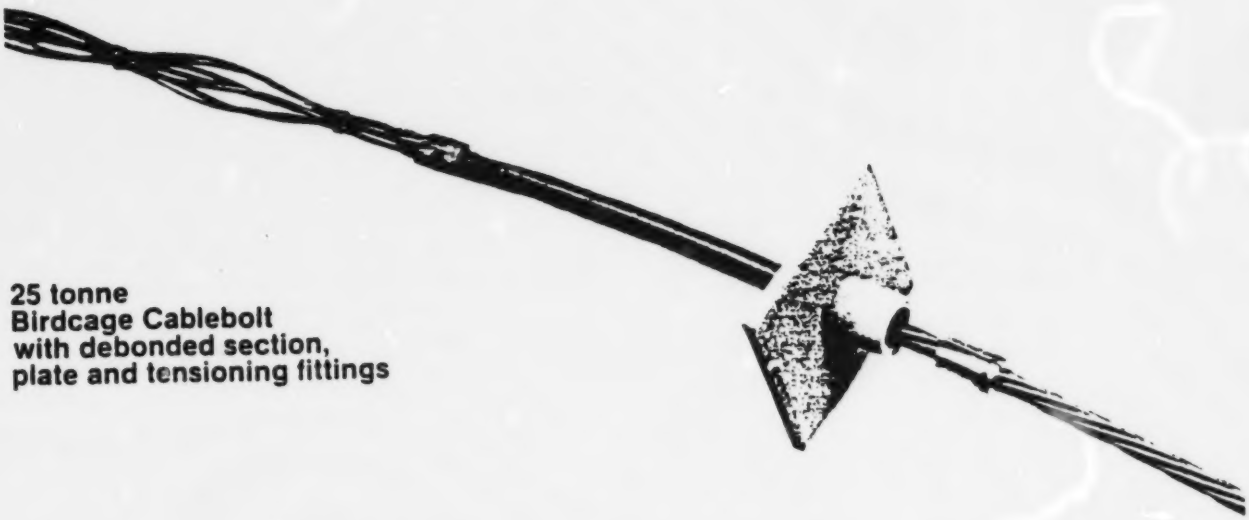


Figure 20: Birdcage cable bolt (Rock Engineering Pty)

5.0 DISCUSSION

It is observed by most researchers that cable failure is generally in the form of large blocks slipping off the cable and not the actual severing of the tendon. Small thin blocks with a short bonded length simply slide off the cables due to a low bond strength per block. Blocks between cables break free in the low stress condition that exists near the exposed wall and as a result slide off the cable. Therefore the improved bond strength would maximize the overall support capability of the cable bolt. This is partly achieved through anchoring the face of the opening to the cable by means of barrel and wedge/strap assemblies. The use of fittings to maximize the load transfer between the rock mass and the cable would enhance the bond strength. Tests are presently being conducted by the U.S.B.M (Goris,1989) on the effect of the use of these barrels. In addition, a program of research in the use of cables for open stope support is being conducted by ADMIRA (Australia).

It is concluded by all researchers that a need is required to maintain a high standard of grout preparation and cable cleanliness. Many failures of cable supports can be attributed to poor installation practice. Debonding of cables is an area which has shown to be successful (Fuller, 1983) in controlling dilution and maximizing sill pillar support.

Failures in stope backs are largely structurally controlled and cable design is based upon delineating the wedge and supporting it. Cable bolt design in stope walls is largely empirical and limited by access. The industry requires a reliable and quantitative calculation approach to the design of cable support in open stope operations. This will only be achieved through documented use and the assessment of the overall performance.

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APPENDIX II

Blast Modelling - 550 20JN Stope

APPENDIX II

BLAST MODELING

1.0 Introduction

This appendix describes the finite element modeling of velocity records of blasts detonated in the 20JN stope at the Ruttan Mine as part of experiments to determine the effects of blasting on cable bolting.

The locations of the blasts are shown on Figure II-1. Velocity time histories from two blasts, designated 3X and 5X and detonated in the ore zone between elevations 1050 m and 1080 m, were recorded on the floor of a drift located at elevation 1092 m. The vertical components of these velocity records are shown on Figure II-2.

The explosive used was LOMEX, a CIL product. Its properties are given in Table II-1. The peak pressure is computed using the formula given in the Pit Slope Manual [1], assuming the charge completely filled the borehole.

The host rock is an acid sedimentary sequence with some cherty limestone. The ore is a sulphide with occasional plugs of andesite. Properties of these materials are given in Table II-2.

2.0 Modeling Methodology

The blasts were composed of several delays, each of which could be modeled individually. Wave propagation due to a blast was modeled by the finite element program described in [2].

The geometry of the blasts and recording location is three dimensional. This is particularly true of Blast 5X. However, the wave propagation program is only capable of modeling two dimensional geometries. Therefore some approximations had to be made.

It was first decided to abandon attempts to model any delays of Blast 5X. Individual blastholes of this blast were located in five rings as shown on Figure II-1. The recording location does not lie directly above rings 1 to 5, making it difficult to define a two dimensional cross-section, which includes the blast, along the length of the stope. Also the spatial location of each cap within the rings does not lend itself to a two dimensional approximation.

Blast 3X was located in rings 7, 8 and 9 shown on Figure II-1. The recording location was located almost directly above this blast. This geometry lends itself to a two dimensional idealization. A cross-section of ring 9 is shown on Figure II-3.

Attention was focused on the blast detonated by cap number 23 of Blast 3X which was located in hole number 7 shown in Figure II-3. This portion of the blast produced the waveform shown in the box on Figure II-2. The waveform is shown on Figure II-4. The nominal firing time of the delay was 1.675 seconds.

In order to model this waveform, a finite element model of the cross-dection shown on Figure II-3 was constructed. The details of this model are discussed in the next section.

3.0 Finite Element Model

The finite element mesh shown on Figure II-5 was used. This mesh consists of 1333 nodes and 1260 elements. The geometry of the ore zone in the stope and the surrounding host rock was approximated as shown. The ore and host rock were assigned the properties of in-situ materials given in Table II-2.

The mesh was regular with 31 nodes along the stope width and 43 nodes in the vertical direction. The drift between elevations 1077 m and 1080 m was modeled by fixing the nodes within the rectangular area shown and assigning the elements within the area zero modulus. Since this is the last delay of the blast, it was assumed that the irregular area between elevations 1050 m and 1063 m where previous delay blasts had occurred was air. This area was also modeled by fixing the nodes within the area and assigning zero modulus to the elements. The floor of the drift in which the blast was recorded is represented by the top surface of the model at elevation 1092 m.

The nodes at the lower left and lower right corners were fixed. All other nodes outside the air zones were free to move in both directions. The vertical component of the velocity was monitored at node 1310 on the top surface of the model, as shown on Figure II-5. Silent boundaries (see [2]) were used on the left and right sides of the model to avoid spurious reflections from these artificial boundaries.

The blast was modeled as a time-dependent vertical point load at node number 11 as shown on Figure II-5. The pulse shape is shown on Figure II-6. The peak pressure of this pulse is approximately 90 MPa at time 4.0×10^{-4} s. This pressure is much lower than the quoted blasthole pressure of the explosive (LOMEX) but is comparable to the compressive strength of the ore rock. One possible explanation for the low blasthole pressure is that the actual blast was not completely confined in the blasthole due to the presence of the blasted ore between elevations 1050 m and 1063 m.

The time integration scheme used to solve the system of differential equations resulting from the finite element discretization is explicit. Such schemes are conditionally stable, the condition being that the time step be less than the smallest time required for compressional waves to traverse an element of the mesh. From Table II-2 the fastest compressional velocity is 4581 m/s. The elements are 1 m square so that the critical time step is $1/4581 \approx 2.2 \times 10^{-4}$ s. To ensure stability, a time step of 10^{-4} s was selected. The number of time steps was 1000.

4.0 Results

The computed vertical component of velocity at node 1310 is shown on Figure II-7. The first arrival of the computed waveform occurs much earlier than that of the observed waveform. The possible explanations for this are:

1) The detonation time of the cap, 1.675 s, may be in error.

2) The in-situ velocities of the host rock and/or ore may be too high.

3) There may exist three-dimensional or other geometrical effects which effectively decrease wave propagation speed.

Despite this, the shape of the waveform is similar to that observed. By shifting the computed waveform by 0.055 s so that the its first arrival coincides with that of the observed waveform, the comparison shown on

Figure II-8 results.

The low observed values of velocity were of interest. One likely explanation for this is the presence of the drift between elevations 1077 m and 1080 m. This effectively acts as a barrier to wave propagation. After replacing this drift with ore rock, the waveform shown on Figure II-9 is computed at node 1310. The velocities in this waveform are of higher amplitude than those on Figure II-7. The shape of the waveform is also somewhat different than that on Figure II-7. The presence of this drift may be one of the reasons the cable bolts lying between elevation 1080 m and 1090 m were unaffected by the blasts.

5.0 Conclusions and Recommendations

The following conclusions can be made:

1) The comparison between the recorded velocity due to cap 23 of blast 3X and the computed velocity is reasonable, given the number of assumptions involved. However, the computed first arrival time is much earlier than that observed. Also, the peak pressure of the blast had to be significantly lower than the quoted blasthole pressure of the explosive (LOMEX).

2) The low observed values of velocity are likely due to the presence of the drift between elevations 1077 m and 1080 m. This effectively acts as a barrier to wave propagation.

The following recommendations are given. Each has to do with improving the capabilities of the finite element model.

1) The capability to model blasts in three dimensions should be developed. The main reason for this is that geometrical effects seem to play a large part in determining observed amplitudes and waveforms.

2) Better, but simple, methods of representing blast loads in finite element wave propagation models need to be developed. The representations should contain only a few parameters which can be related to physical aspects of the blast. In the common case where waveforms are to be computed at distances greater than the length of the explosive, it would not be necessary to model the details of the blast propagation in the hole, which is a nonlinear phenomenon. However, a pressure time history on the walls of an equivalent cavity (see [2]) could be developed.

3) The explicit solution scheme is efficient from the storage point of view. (All the above calculations were done on an IBM PS/2, Model 80 with 640Kb accessible memory). However, owing to the small time step required for stability, execution times are very long. Implicit schemes, on the other hand, are unconditionally stable, and the time step need only be small enough to ensure accuracy. The storage requirements for implicit schemes are greater and may exceed that available on most microcomputers. However, considerably more memory is available on modern microcomputers and it can be accessed with available compilers.

No particular priority is given to any of the above recommendations. The first and third would be the easier (if time-consuming) to implement.

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2. D. Gordon Dunbar and Associates, 1988. *The Development of a Finite Element Algorithm for Blast Modeling*, Report to Canada Centre for Mineral and Energy Technology.

TABLE II-1

Explosive Properties

LOMEX	Detonation velocity, V:	2000 m/s
	Specific gravity, ρ:	0.53
	Borehole pressure, P:	1517 MPa†

†Borehole pressure is taken from Figure 1(a), Reference 1.

TABLE II-2

Material Properties

E - Young's modulus
ν - Poisson's ratio
ρ - density
S_u - unconfined compressive strength
c_p - compression wave velocity

Rock Type	E (GPa)	ν	ρ (kg/m^3)	S_u (MPa)	c_p (m/s)†
Ore (intact)	88	0.2	2700	81	
Ore (in-situ)	51	0.2	2700		4581
Host (intact)	77	0.2	2700		
Host (in-situ)	23	0.2	2700		3077

†Compressional wave velocity, c_p, is given by the formula

$$c_p = \sqrt{\frac{E(1-\nu)}{(1+\nu)(1-2\nu)\rho}}.$$

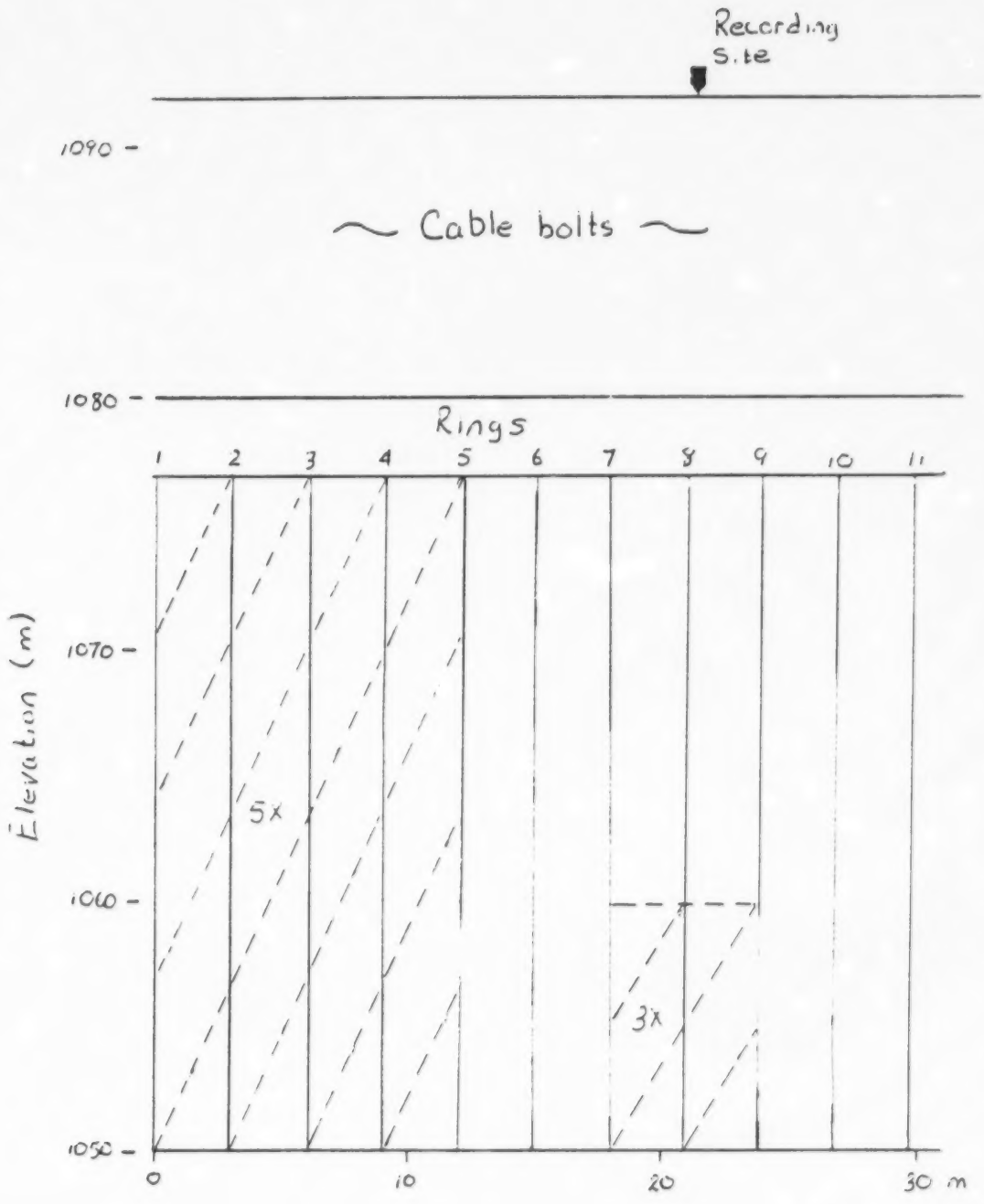

Figure II-1
Locations of blasts 3X and 5X along 20JN stope, looking northwest.

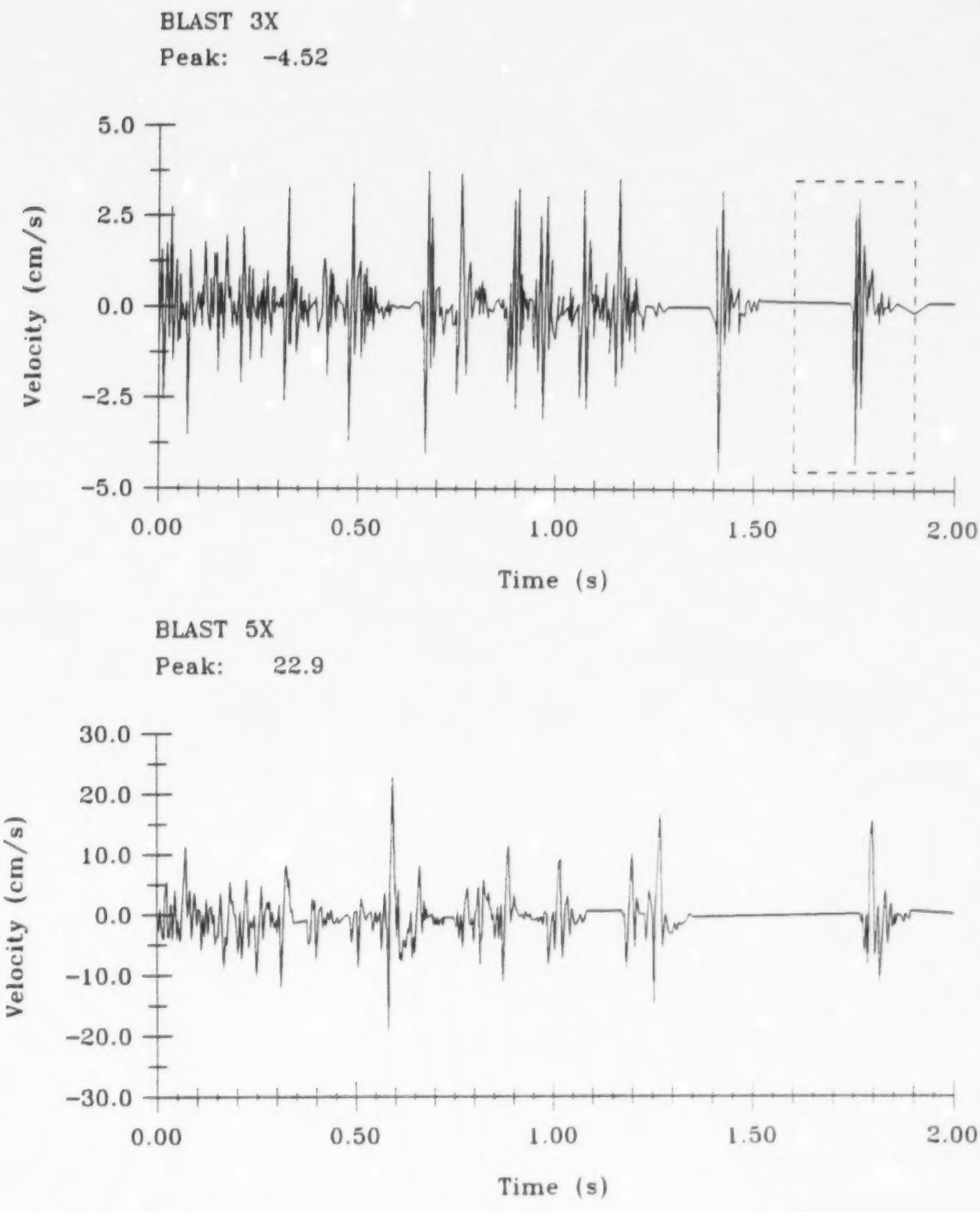

Figure II-2

Recorded vertical components of velocity at elevation 1092 m due to blasts 3X and 5X. Dashed box in blast 3X indicates waveform selected for modeling.

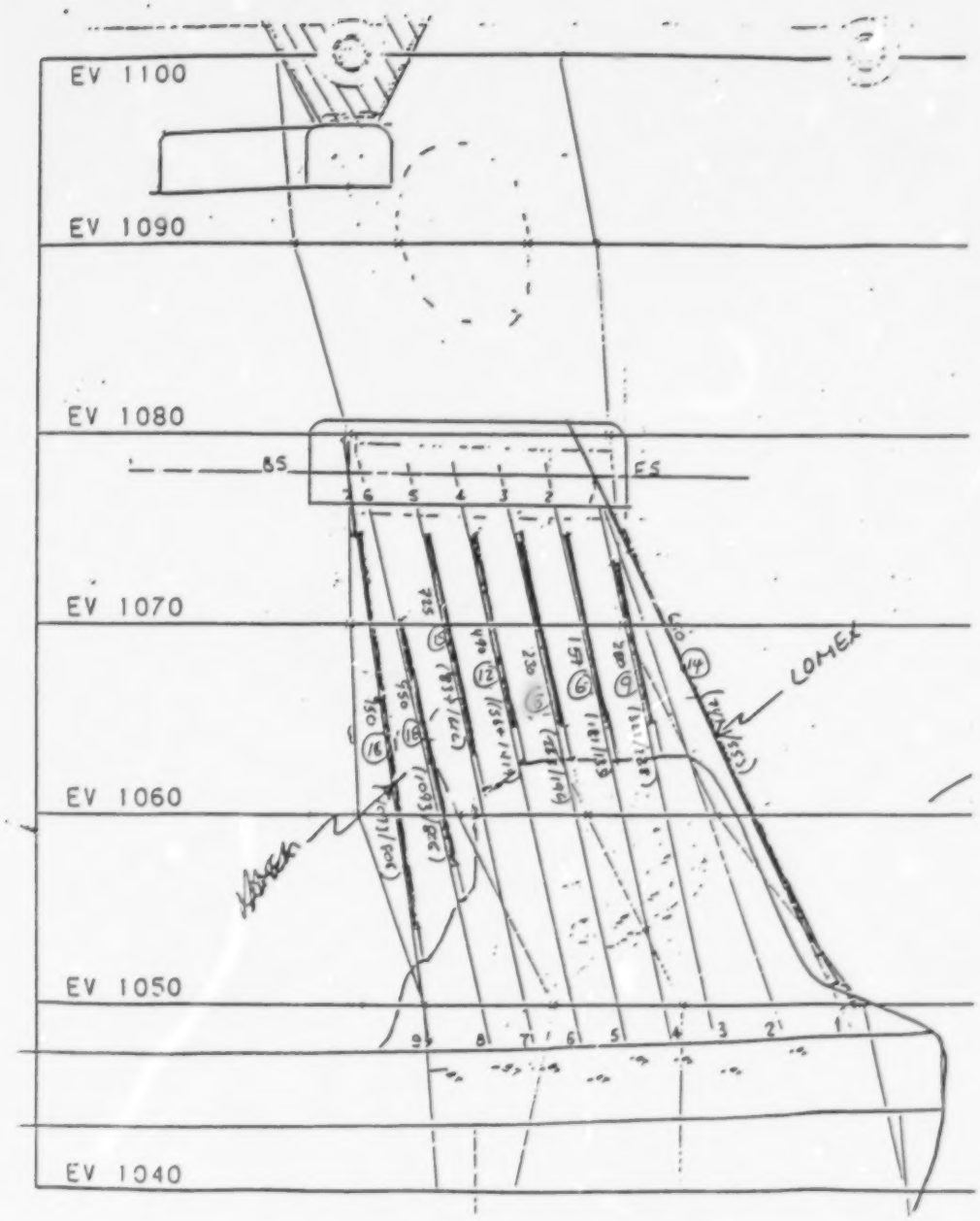

Figure II-3
Ring 9, cross-section looking southwest, showing blasthole locations.

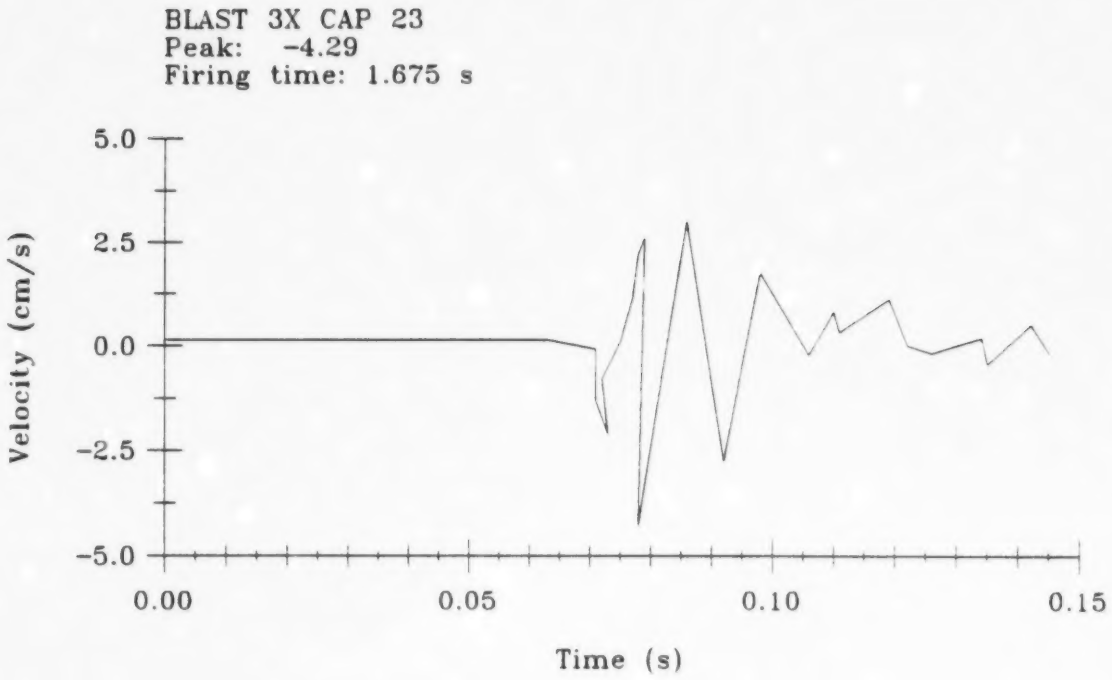

Figure II-4
Recorded waveform, cap 23 of blast 3X (see box on Figure II-2).

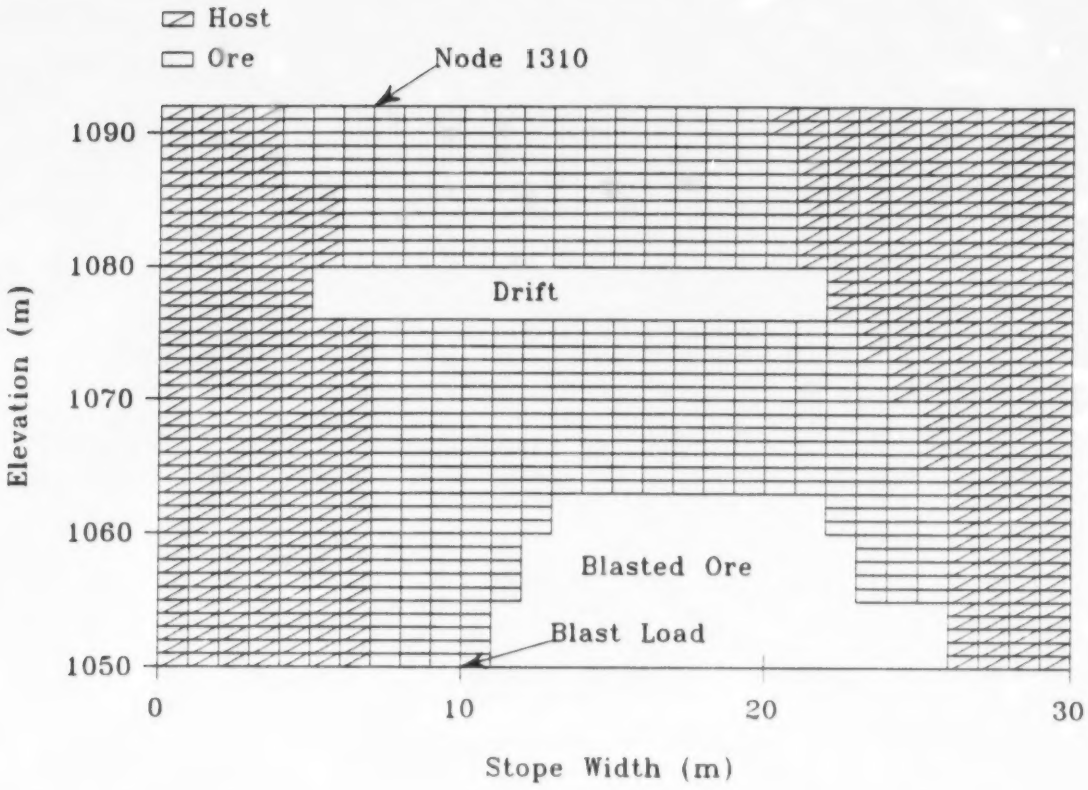

Figure II-5
Finite element model of cross-section shown on Figure II-3.

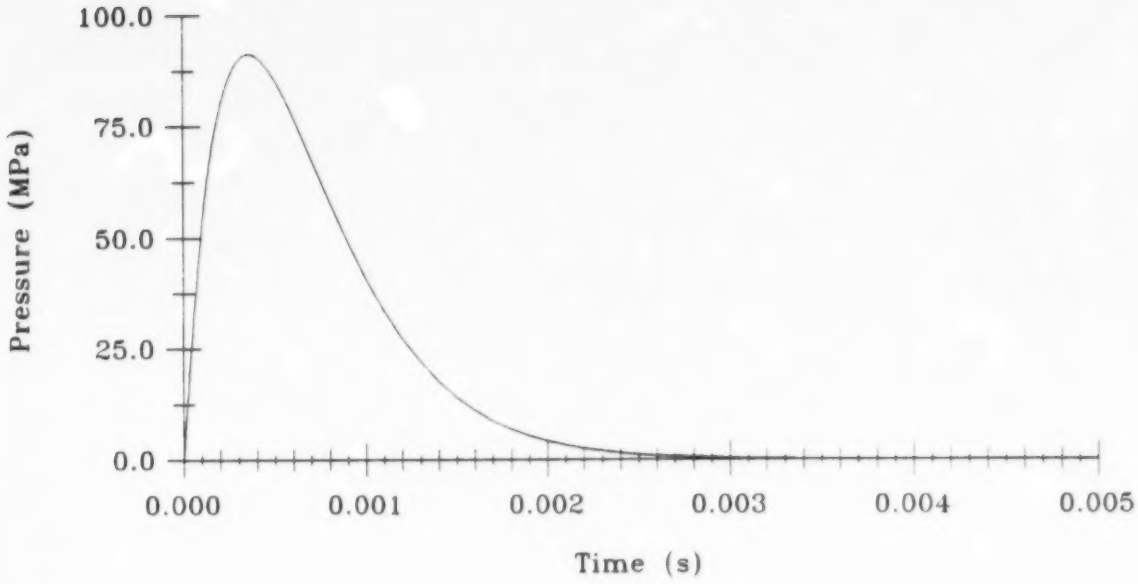

Figure II-6
Blast pulse pressure time history, $p = ae^{-\beta t}\sin(\omega t)$ MPa, where $a = 1100.0$, $\beta = 2763.1$, and $\omega = 200\pi$.

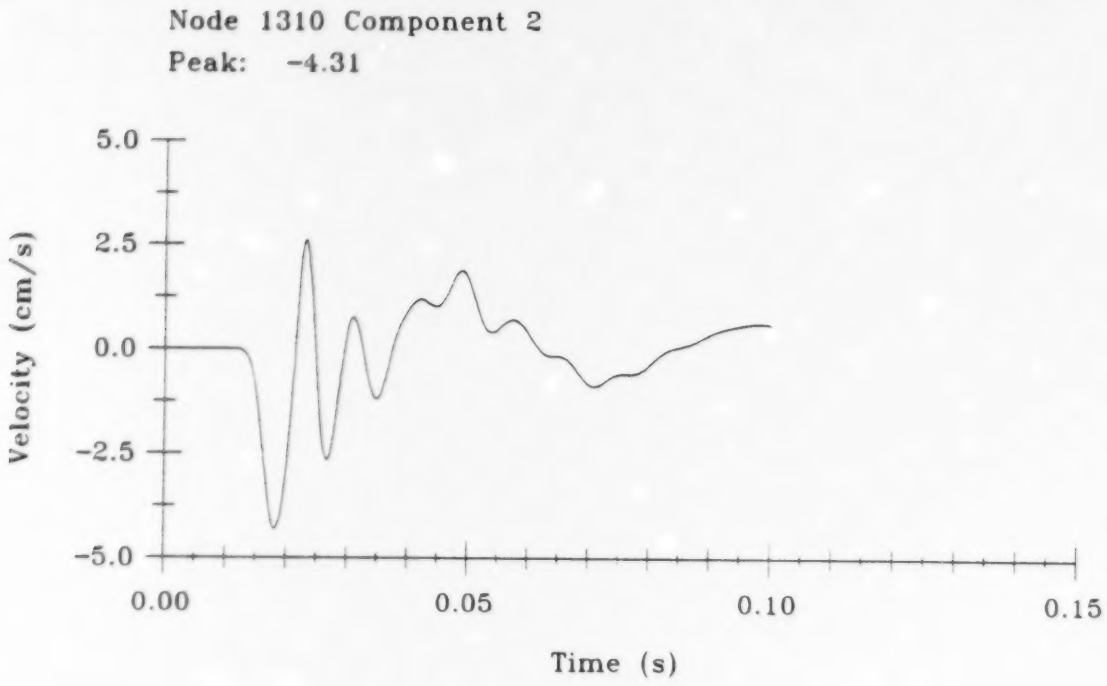

Figure II-7

Computed velocity waveform. Vertical component at node 1310 of finite element model shown on Figure II-5.

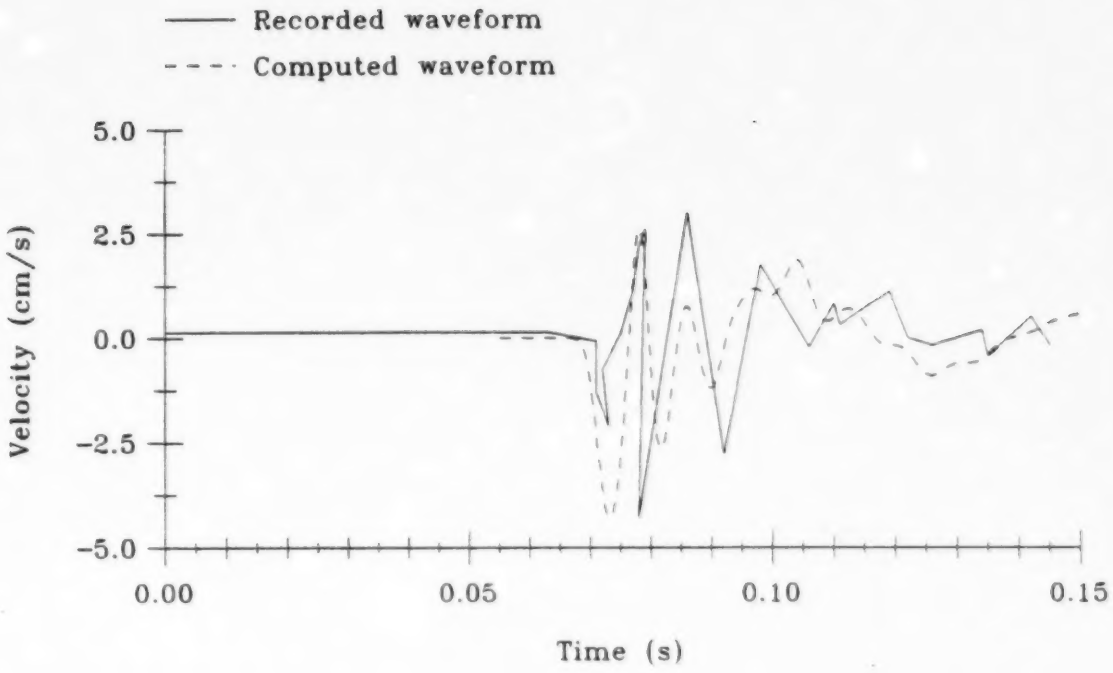

Figure II-8
Comparison of computed velocity waveform, time-shifted by 0.055 s, with recorded waveform.

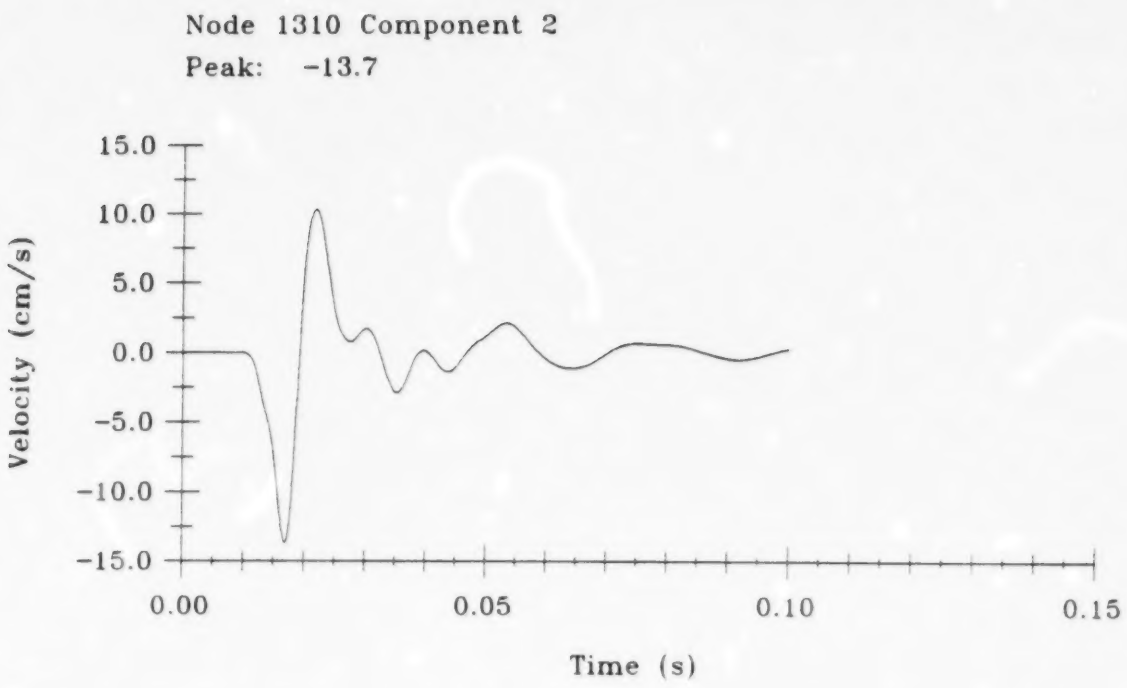

Figure II-9

Computed velocity waveform. Vertical component at node 1310 of finite element model shown on Figure II-5 after replacing drift at elevation 1080 m with ore.

APPENDIX III

TENSMEG - 70 Brochure

SUPPORTEK INC

Instrumentation and consulting services on rock supporting techniques

TENSION MEASURING GAUGE FOR CABLE BOLTS

MODEL TENSMEG-70

FEATURES

- Low cost gauge
- Any number installed along length of cable bolt
- Overall diameter when installed: 1 1/4 inch (31.2 mm)
- For short term and long term instrumentation or monitoring in surface and underground mines, rock slopes, dams, prestressed concrete...
- Monitoring of tension loads beyond yield limit of cable up to the breaking load
- Can be manufactured to suit other shapes than standard 5/8 inch (15.9 mm) cable
- High sensitivity of readout unit
- Can be connected to a data acquisition system

DESCRIPTION

Tension measuring gauge mounted on a cable bolt shown with plastic protective tubings for two end anchors. Electrical cable for plugging to readout is connected to gauge

The Tension Measuring Gauge TENSMEG-70 was developped at Laval University (Quebec city, Canada). It is based on a thin 2,4 m resistance wire folded back three times between two end anchors for a total gauge length of 0.7 m.

The TENSMEG-70 is easily mounted on a cable by gluing one end anchor, then winding the wires in the grooves between the strands of the cable and finally gluing the second end anchor.

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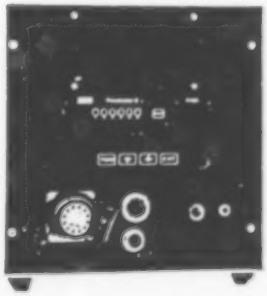
Fax: 418-694-9679

A special and very simple tensionning tool (not shown) is used to slightly pre-tension the gauge and ensure accuracy of readings. The working principle of the TENSMEG-70 is based on the change of resistance of the resistance wire caused by cable elongation. Reading out is conducted on a five channel readout unit, or the gauges can be connected to a standard data acquisition system.

Calibration tests were conducted in a traction load frame on 5/8 inch cable bolts instrumented with a TENSMEG-70 and grouted in a 50 mm steel tube. They showed an excellent linear relationship between readout values and tension applied to the cable. The TENSMEG-70 was also tested in underground and surface mines.

FIVE CHANNEL READOUT UNIT

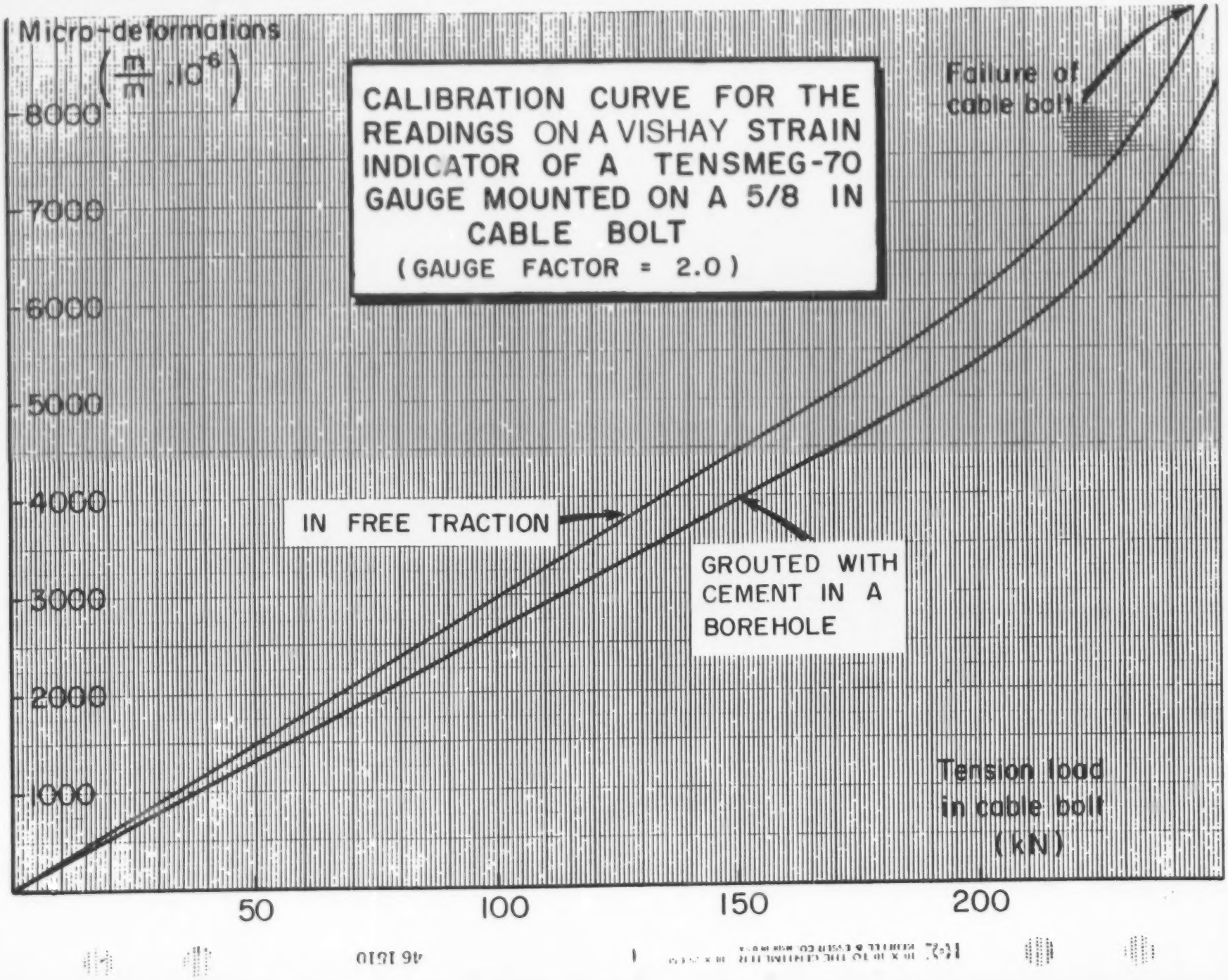


APPLICATIONS

The Tension Measuring Gauge TENSMEG-70, a number of which being installed along the length of the cable bolt, provides a measure of the tension load which is building in the cable. Similarly to a borehole extensometer, the TENSMEG-70 can be used either as an instrumentation for the purpose of observing the rock mass behaviour or for short term or long term monitoring purposes, looking for example at the point where the tension load in the cable is approaching its breaking strength, typically 59000 lb (260 kN) for a 5/8 inch cable bolt. In addition, various monitoring applications can also be envisaged in reinforced and prestressed concrete, dams, road cuts, rock slopes...

ACCESSORIES

1. Available on purchase or rental basis:
   - Five channel readout unit
   - Tensionning tool
   - Opening tool for inserting end anchors
   - Clamping tool for gluing end anchors
2. Electrical cable (specify total length and depth of gauges in borehole), connections, glue.


Micro-deformations $\left(\frac{m}{m} \cdot 10^{-6}\right)$
8000
7000
6000
5000
4000
3000
2000
1000
CALIBRATION CURVE FOR THE READINGS ON A VISHAY STRAIN INDICATOR OF A TENSMEG-70 GAUGE MOUNTED ON A 5/8 IN CABLE BOLT
(GAUGE FACTOR = 2.0)
Failure of cable bolt
IN FREE TRACTION
GROUTED WITH CEMENT IN A BOREHOLE
Tension load in cable bolt (kN)
50
100
150
200